CHAPTER 7 – STORM DRAIN DESIGN

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7.1 Background

This chapter provides guidance on storm drain design and analysis. A storm drain is the portion of the highway drainage system that receives surface water through inlets and conveys the water through conduits to an outfall. It is composed of different lengths and sizes of pipe or conduit connected by structures. A section of conduit connecting one inlet or structure to another is termed a "segment" or "run." The storm drain is usually a circular pipe, but can also be a box or other enclosed conduit shapes. Structures include inlet structures (excluding the actual inlet opening), access holes, junction chambers, and other miscellaneous structures.(1)

Where feasible, the storm drains shall be designed to avoid existing utilities. Attention shall be given to the storm drain outfalls to ensure that the potential for erosion is minimized. Drainage system design should be coordinated with the proposed staging of large construction projects to maintain an outlet throughout the construction project.

For a general discussion of policies and guidelines for storm drainage, the designer is referred to References (2) and (3). For more design and engineering guidance, refer to AASHTO, *Highway Drainage Guidelines*, Chapter 9,(4) and HEC 21(5) and HEC 22 (www.fhwa.dot.gov\bridges\hydpub.htm).(1)

7.2 Design Guidelines

The following guidelines are to be followed unless the GDOT drainage committee grants an exception.

7.2.1 Design Frequency

- Longitudinal pipes for storm drains shall be designed to accommodate the 10-year frequency storm.

- Storm drain systems shall be designed to accommodate the 50-year storm in areas where the flow has no outlet except through the storm drain system, and failure of the drainage system could result in flooding or inundation of the roadway such as low points in cuts or depressed roadways. If the flow can overtop the curb and escape overland, the 50-year design criterion is not required.

7.2.2 Maximum Structure Spacing

Drainage structures should be spaced to facilitate regular maintenance. Adequate access is required for inspection and cleanout of storm drain systems. The following guides are the maximum distances between access points within a closed storm drain system:

Pipe Size	Maximum Spacing Interval
$\leq$ 36 in.	400 ft
> 36 in.	600 ft

7.2.3 Minimum Conduit Criteria

- Minimum pipe size to be used shall be 18 inches. Specific projects may dictate a minimum pipe size larger than 18 inches such as in flat terrain.

- Clearances: All conduits (pipes, boxes, etc.) shall have a minimum cover of 1 ft. The minimum roadway clearance over a conduit shall be 1 ft measured from the bottom of the pavement structure to the exterior crown. Underground utilities shall have a minimum clearance of 0.5 ft from the exterior crown of the conduit.

- Generally, storm drains should be designed to provide a velocity of at least 3 ft/s, if possible, to ensure that the pipe is self-cleaning. See section 7.8.5 for additional information on grades required to obtain a 3 ft/s velocity.

7.3 Design Approach

The design of a storm drain system evolves as a project develops. The design process composes two separate procedures:

- After all inlets have been located, a preliminary system is developed where the storm drain is sized and losses at the structures are estimated.

- Next, the hydraulic grade line must be calculated to ensure that the system will work.

The general process is as follows:

1. Collect data
2. Coordinate with other agencies
3. Prepare preliminary sketch
4. Plan layout of storm drain system:
    - Locate main outfall
    - Determine direction of flow
    - Locate existing utilities
    - Locate connecting mains
    - Locate access holes
5. Preliminary size storm drain system conduits
6. Develop hydraulic grade line
7. Revise preliminary design as needed
8. Prepare the plan
9. Document design

7.4 Data Collection

The designer should be familiar with land-use patterns, the nature of the physical development of the area(s) to be served by the storm drainage system, the stormwater management plans for the area, and the ultimate pattern of drainage (both overland and by storm drains) to some existing outfall location. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available. Furthermore, there should be an understanding of the nature of the outfall because it usually has a significant influence on the storm drainage system. In environmentally sensitive areas, there may be water quality requirements to consider as well.

7.5 Coordination With Other Agencies

Cooperative storm drain projects with cities and municipalities may be beneficial where both a mutual economic benefit and a demonstrated need exist. Early coordination with the governmental entity involved is necessary to determine the scope of the project. Each cooperative project may be initiated by a resolution adopted by the governing body of the municipality either (1) requesting the improvements and/or indicating its willingness to share the cost of a State project, or (2) indicating the municipality's intention to make certain improvements and requesting State cost participation in the municipal project.

7.6 Preliminary Sketch

Preliminary sketches or schematics, featuring the basic components of the intended design, are useful. Such sketches should indicate:

- Watershed areas and land use
- Existing drainage patterns
- Plan and profile of the roadway
- Roadway cross section
- Typical sections
- Street and driveway layout with respect to the project roadway
- Underground utility locations and elevations
- Locations of proposed retaining walls, bridge abutments, and piers
- Logical inlet and access hole locations
- Preliminary lateral and trunk line layouts
- Clear definition of the outfall location and characteristics

This sketch should be reviewed with the traffic staging plans and soils recommendations for areas that are incompatible with required construction staging. With this sketch or schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments and refinements.

Unless the proposed system is very simple and small, the designer should develop a preliminary plan as described above.

7.7 Plan Layout of Storm Drain System

- Locate the storm drain to avoid conflicts with utilities, foundations or other obstacles
- Minimize the depth of the storm drain within the minimum cover required
- Coordination with utility owners during the design phase is necessary to determine if an adjustment to the utilities or the storm drain system is required, the location of the storm drain may affect construction activities and phasing
- Storm drain should be located to minimize traffic disruption during construction and future maintenance
- Dual trunk lines along each side of the roadway may be used in some cases where it is difficult or more costly to install laterals
- Do not locate the storm drains under the pavement but on public property such as under or behind sidewalks whenever possible.
- Easements may be obtained if required
- Temporary drainage may be needed to avoid increases in flood hazards during construction

7.7.1 Outfall Concerns

In the design of a storm drain system, establish the location of the outlet. This outlet becomes one of the control points that will influence the grade and the subsequent design of the system. Always strive for a gravity flow system. Pumping stations are to be avoided except in extreme circumstances and never proposed without consultation with the GDOT.

Since highway systems may increase peak discharge and volume due to increases in the impervious area and decreases in the time of concentration or lag time, accumulation or diversion of flow may also result in an increase in runoff at storm drain outfalls. The channel stability of outfall channels/storm drain systems must be assessed especially when there are significant changes in discharges due to highway projects or developments.

Detention basins may be used to reduce flooding caused by the increased runoff (see Chapter 8 for more information on detention basin design). These basins may be designed to keep the post development runoff the same as the predevelopment runoff. Limiting detention peak outflow only for large storm or flood events could result in uncontrolled increases in runoff for the smaller more frequent storms that may be undesirable. Therefore, the basins may need to be designed to accommodate a range of floods, from the 2- to the 100-year event.

Outfall channels may also be subject to erosion from the higher more concentrated flows. Therefore, erosion protection and or use of energy dissipators may be required (see Chapters 11 and 12).

7.8 Preliminary Design of Storm Drainage Systems

As previously noted, storm drain design consists of two general processes: a preliminary layout of the system and the check of the hydraulic grade line (HGL). The preliminary layout begins at the upstream end of the system and includes the initial sizing of the conduits and preliminary estimate of the energy losses across the structures. After the system has been initially laid out, the HGL check is made starting at the downstream end of the system working back upstream. The actual losses may then be established more accurately by computing all losses including the exit loss, friction losses, structure losses, etc.

7.8.1 Hydraulics of Storm Drain Systems

Hydraulic design of storm drainage systems requires an understanding of basic hydrologic and hydraulic concepts and principles. Hydrologic concepts were discussed in Chapter 4. Important hydraulic principles include flow classification, conservation of mass, conservation of momentum, and conservation of energy. Some of these elements were introduced in Chapter 3. The following sections assume a basic understanding of these topics.

7.8.2 Flow Type Assumptions

The design procedures assume that flow within each storm drain segment is steady and uniform. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant.

In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption.

7.8.3 Open Channel vs. Pressure Flow

Two design philosophies exist for sizing storm drains under the steady uniform flow assumption:

- The first is referred to as open channel or gravity flow design. To maintain open channel flow, the segment must be sized so that the water surface within the conduit remains open to atmospheric pressure. For open channel flow, flow energy is derived from the flow velocity (kinetic energy), depth (pressure), and elevation (potential energy). If the water surface throughout the conduit is to be maintained at atmospheric pressure, the flow depth must be less than the height of the conduit.

- The second is pressure flow design which requires that the flow in the conduit be at a pressure greater than atmospheric. Under this condition, there is no exposed flow surface within the conduit. In pressure flow, flow energy is again derived from the flow velocity, depth, and elevation. The significant difference here is that the pressure head will be above the top of the conduit, and will not equal the depth of flow in the conduit. In this case, the pressure head rises to a level represented by the hydraulic grade line.

For a given flow rate, design based on open channel flow requires larger conduit sizes than those sized based on pressure flow. While it may be more expensive to construct storm drainage systems designed based on open channel flow, this design procedure provides a margin of safety by providing additional headroom in the conduit to accommodate an increase in flow above the design discharge. This factor of safety is often desirable since the methods of runoff estimation are not exact, and once placed, storm drains are difficult and expensive to replace.

However, there may be situations where pressure flow design is necessary. In some cases it may be necessary to use an existing system that must be placed under pressure flow to accommodate the proposed design flow rates. In fact, some sites will dictate that the system will operate in pressure flow. For example, when the tailwater is high, the system will operate in pressure flow regardless of how the system is designed.

In light of the previous discussion, storm drains are to be sized based on gravity flow criteria at full flow or near full. Designing for full flow is a conservative assumption since the peak flow actually occurs at 93% of full flow. When pressure flow does occur, special emphasis should be placed on the proper design of the joints to prevent future problems with leakage and subsequent sinkholes.

7.8.4 Sizing of Storm Drain

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's Equation.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{7.1}$$

where:

V	=	Mean velocity of flow, ft/s
n	=	Manning's roughness coefficient
R	=	Hydraulic radius, ft – area of flow divided by the wetted perimeter (A/WP)
S	=	Slope of the energy grade line, ft/ft

In terms of discharge, the above formula becomes:

$$Q = VA = \frac{1.486}{n} AR^{2/3} S^{1/2} \tag{7.2}$$

where:

Q	=	Flow rate, ft^3/s
A	=	Cross sectional area of flow, ft^2

These two equations will work for any shape of conduit. However, for circular storm drains flowing full, R = D/4 and Equations 7.1 and 7.2 become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \tag{7.3}$$

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{7.4}$$

where:

D	=	Diameter of pipe, ft

Equation 7.4 may be rearranged to solve for the diameter directly and becomes:

$$D = \left(\frac{Q\ n}{0.463\ S^{0.5}} \right)^{0.375} \tag{7.5}$$

A nomograph solution of Manning's Equation for full flow in circular conduits is presented in Chart 7.1 in Appendix 7-B.

Representative values of the Manning's coefficient for various storm drain materials are provided in Table 7.1. It should be remembered that the values in the table are for new pipe tested in a laboratory. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

The hydraulic elements graph in Figure 7.1 is provided to assist in the solution of the Manning's equation for part full flow in storm drains. The hydraulic elements chart shows the relative flow conditions at different depths in a circular pipe and makes the following important points:

1. Peak flow occurs at 93% of the height of the pipe. This means that if the pipe is designed for full flow, the design will be slightly conservative.
2. Velocity in a pipe flowing half-full is the same as the velocity for full flow.
3. Flow velocities for flow depths greater than half-full are greater than velocities at full flow.

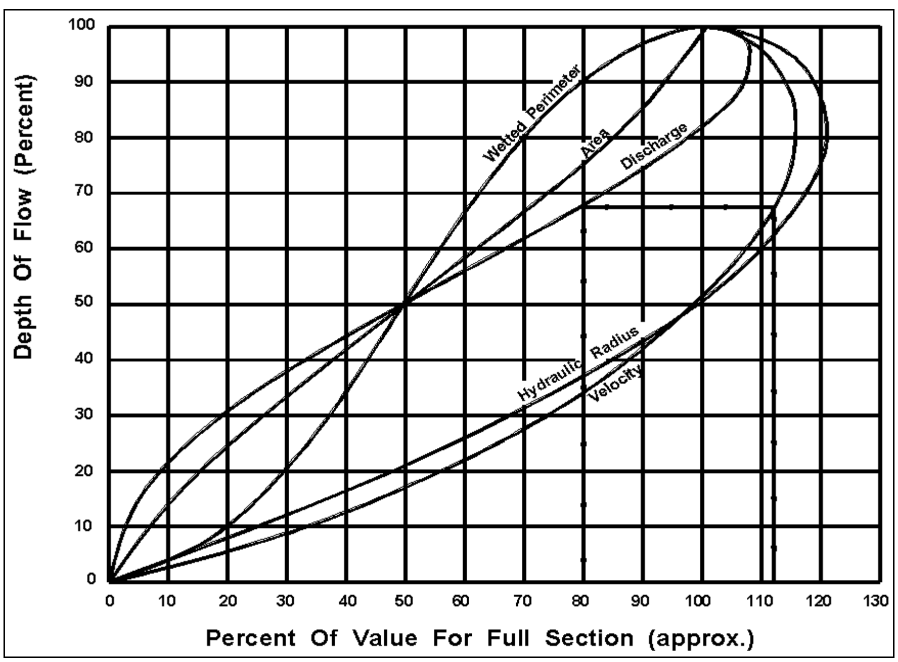


Figure 7.1. Hydraulics elements chart.

Table 7.1. Manning's Coefficients for Storm Drain Conduits.*(1)

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	Smooth	0.010-0.011
Concrete Boxes	Smooth	0.012-0.015
Spiral Rib Metal Pipe	Smooth	0.012-0.013
Corrugated Metal Pipe, Pipe-Arch and Box (Annular or Helical Corrugations -- see Figure B-3 in Reference 2, Manning's n varies with barrel size)	68 by 13 mm 2-2/3 by 1/2 in Annular	0.022-0.027
	68 by 13 mm 2-2/3 by 1/2 in Helical	0.011-0.023
	150 by 25 mm 6 by 1 in Helical	0.022-0.025
	125 by 25 mm 5 by 1 in	0.025-0.026
	75 by 25 mm 3 by 1 in	0.027-0.028
	150 by 50 mm 6 by 2 in Structural Plate	0.033-0.035
	230 by 64 mm 9 by 2-1/2 in Structural Plate	0.033-0.037
Corrugated Polyethylene	Smooth	0.009-0.015
Corrugated Polyethylene	Corrugated	0.018-0.025
Polyvinyl chloride (PVC)	Smooth	0.009-0.011

*NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.

4. As the depth of flow drops below half-full, the flow velocity drops off rapidly.

The shape of a storm drain conduit also influences its capacity. Although most storm drain conduits are circular, a significant increase in capacity can be realized by using an alternate shape.

7.8.5 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 3 ft/s at design flow. For very flat grades, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to ensure that there is sufficient velocity in all drains to deter settling of particles. Minimum slopes required for a velocity of 3 ft/s can be calculated by the Manning's formula, or obtained using Table 7.2:

$$S = 2.87\left[\frac{nV}{D^{0.67}}\right]^2 \tag{7.6}$$

Table 7.2. Minimum Slopes Necessary to Ensure 3 ft/s in Storm Drains Flowing Full.

Pipe Size (in)	Full Pipe (ft^3/s)	Minimum Slopes (ft/ft)		
		n = 0.012	n = 0.013	n = 0.024
8	1.05	0.0064	0.0075	0.0256
10	1.64	0.0048	0.0056	0.0190
12	2.36	0.0037	0.0044	0.0149
15	3.68	0.0028	0.0032	0.0111
18	5.30	0.0022	0.0026	0.0087
21	7.22	0.0018	0.0021	0.0071
24	9.43	0.0015	0.0017	0.0059
27	11.93	0.0013	0.0015	0.0051
30	14.73	0.0011	0.0013	0.0044
33	17.82	0.00097	0.0011	0.0039
36	21.21	0.00086	0.0010	0.0034
42	28.86	0.00070	0.00082	0.0028
48	37.70	0.00059	0.00069	0.0023
54	47.71	0.00050	0.00059	0.0020
60	58.90	0.00044	0.00051	0.0017
66	71.27	0.00038	0.00045	0.0015
72	84.82	0.00024	0.00040	0.0014

7.8.6 Access Holes

Access holes are used to provide entry to continuous underground storm drains for inspection and cleanout. Some agencies use grate inlets in lieu of access holes when entry to the system can be provided at the grate inlet, so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where access holes should be specified are:

- Where two or more storm drains converge
- At intermediate points along tangent sections
- Where pipe size changes
- Where an abrupt change in alignment occurs
- Where an abrupt change of the grade occurs

Access holes should not be located in traffic lanes; however, where it is impossible to avoid locating an access hole in a traffic lane, care should be taken to ensure that it is not in the normal vehicular wheel path.

Spacing of Access Holes

The spacing of access holes should be in accordance with Section 7.2.2:

Pipe Size	Maximum Spacing Interval
≤ 36 in.	400 ft
> 36 in.	600 ft

7.8.7 Curved Alignment

Curved storm drains are permitted where necessary. Long-radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 48 in and larger. Short-radius bend sections are also available and can be used if there is not room for the long-radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Using large access holes solely for changing direction may not be cost effective on large-size storm drains.(9)

7.8.8 Energy Loss Estimation for Preliminary Layout

The approximate method for computing losses at access holes or inlet structures involves multiplying the velocity head of the outflow pipe by a coefficient as represented in Equation 7.7. Applicable coefficients (K_{ah}) are tabulated in Table 7.3. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations, as demonstrated in the example problem. However, this method is used only in the preliminary design process and should not be used in the EGL calculations. For calculation of the HGL, a more detailed and precise procedure will be used.

$$H_{ah} = K_{ah}\left(\frac{V_O^2}{2g}\right) \qquad (7.7)$$

where:

H_{ah} = Estimated energy loss (head loss) across the structure, ft
K_{ah} = Head loss coefficient as given in Table 7.3
V_o = Velocity of flow leaving structure in outflow pipe, ft/s
g = Acceleration of gravity (32.2 ft/s/s)

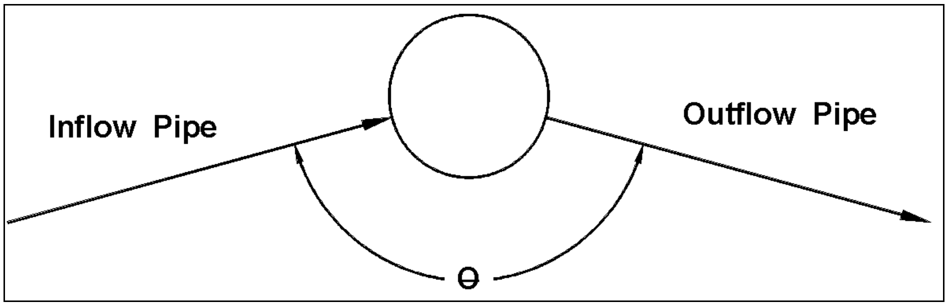


Figure 7.2. Interior angle.

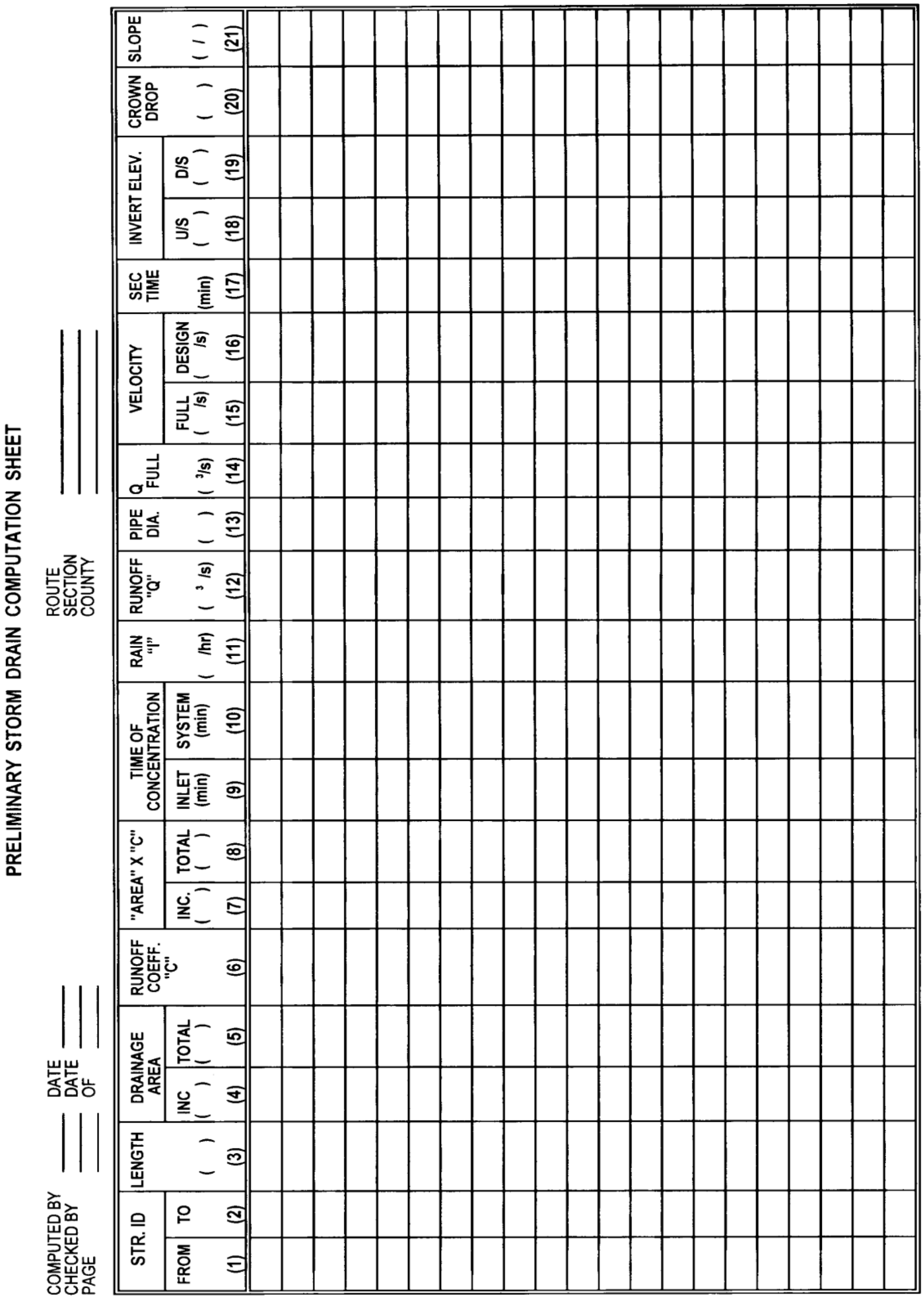
Table 7.3. Head Loss Coefficients.(1)	
	K_{ah}
Inlet - straight run	0.50
Inlet - angled through	
90E	1.50
120E	1.25
135E	1.10
157.5^{o}	0.70
Manhole - straight run	0.15
Manhole - angled through	
90E	1.00
120E	0.85
135E	0.75
157.5E	0.45

7.8.9 Preliminary Layout

The preliminary design of storm drains can be accomplished by using the following steps and the preliminary storm drain computation sheet provided in Figure 7.3. This procedure assumes that each storm drain will be initially designed to flow full under gravity conditions. The designer must recognize that when the steps in this section are complete, the design is only preliminary. Final design is accomplished after the energy grade line and hydraulic grade line computations have been completed (See Section 7.10).

Step 1 Prepare a working plan layout and profile of the storm drainage system establishing the following design information as outlined earlier in this Chapter:

a. Location of storm drains
b. Direction of flow
c. Location of access holes and other structures
d. Number or label assigned to each structure
e. Location of all existing utilities (water, sewer, gas, underground cables, etc.)



PRELIMINARY STORM DRAIN COMPUTATION SHEET

COMPUTED BY ______ DATE ______
CHECKED BY ______ DATE ______
PAGE ______ OF ______

ROUTE ______
SECTION ______
COUNTY ______

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	( )	INC ( )	TOTAL ( )		INC. ( )	TOTAL ( )	INLET (min)	SYSTEM (min)	(/hr)	(³ /s)	( )	(³/s)	FULL (/s)	DESIGN (/s)	(min)	U/S ( )	D/S ( )	( )	(/)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 7.3. Preliminary Storm Drain Computation Sheet.

Step 2 Determine the following hydrologic parameters for the drainage areas tributary to each inlet to the storm drainage system:

a. Drainage areas
b. Runoff coefficients
c. Travel time

Note: The rate of discharge at any point in the storm drainage system is not the sum of the inlet flow rates of all inlets above the section of interest. It is generally less than this total. The Rational Method is the most common means of determining design discharges for storm drain design. As previously discussed, the time of concentration is very influential in the determination of the design discharge using the Rational Method.

The designer is usually concerned with two different times of concentration: one for inlet spacing and the other for pipe sizing. The time of concentration for inlet spacing is the time required for water to flow from the most hydraulically distant point of the drainage area contributing only to that inlet. Typically, this is the sum of the times required for water to travel overland to the pavement gutter and along the length of the gutter between inlets. If the total time of concentration to the upstream inlet is less than five minutes, a minimum time of concentration of five minutes is used as the duration of rainfall. The time of concentration for each successive inlet should be determined independently in the same manner as was used for the first inlet (See Chapter 4).

The time of concentration for pipe sizing is defined as the time required for water to travel from the most hydraulically distant point in the watershed contributing to the design point. Typically, this time consists of two components: (1) the time for overland and gutter flow to reach the first inlet, and (2) the time to flow through the storm drainage system to the point of interest

The flow path having the longest time of concentration to the point of interest in the storm drainage system will usually define the duration used in selecting the intensity value in the Rational Method. Exceptions to the general application of the Rational Method exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of the high runoff coefficient (C value) and high intensity resulting from a short time of concentration. If an exception does exist, it can generally be classified as one of two exception scenarios.

The first exception occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this situation occurs, two separate calculations should be made.

- First, calculate the runoff from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.

- Second, calculate the runoff using only the smaller less pervious area. The typical procedure would be followed using the C value for the small less pervious area and the intensity associated with the shorter time of concentration.

The results of these two calculations should be compared and the largest value of discharge should be used for design.

The second exception exists when a smaller less pervious area is tributary to the larger primary watershed. When this scenario occurs, two sets of calculations should also be made.

- First, calculate the runoff from the total drainage area with its weighted C value and the intensity associated with the longest time of concentration.

- Second, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time the peak from the smaller less pervious tributary area is occurring. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, the intensity associated with the time of concentration from the small less pervious area is used. The portion of the larger primary area to be considered is determined by the following equation:

$$A_c = A\left(\frac{t_{C1}}{t_{C2}}\right) \tag{7.8}$$

where:

A_c	=	Most downstream part of larger primary area that will contribute to discharge during the time of concentration associated with smaller, less pervious area
A	=	Area of the larger primary area
t_{c1}	=	Time of concentration of the smaller, less pervious, tributary area
t_{c2}	=	Time of concentration associated with larger primary area as is used in first calculation

The C value to be used in this computation should be the weighted C value that results from combining C values of the smaller less pervious tributary area and the area A_c. The area to be used in the Rational Method would be the area of the less pervious area plus A_c. This second calculation should only be considered when the less pervious area is tributary to the area with the longer time of concentration and is at or near the downstream end of the total drainage area.

Finally, the results of these calculations should be compared, and the largest value of discharge should be used for design.

Step 3 Using the information generated in Steps 1 and 2, complete the following information on the design form for each run of pipe starting with the upstream most storm drain run:

a. "From" and "To" stations, Columns 1 and 2
b. "Length" of run, Column 3
c. "Inc." drainage area, Column 4

This area is the incremental drainage area tributary to the inlet at the upstream end of the storm drain run under consideration.

d. "C," Column 6

The runoff coefficient is for the drainage area tributary to the inlet at the upstream end of the storm drain run under consideration. In some cases a composite runoff coefficient will need to be computed.

e. "Inlet" time of concentration, Column 9

The time required for water to travel from the hydraulically most distant point of the drainage area to the inlet at the upstream end of the storm drain run under consideration

f. "System" time of concentration, Column 10

"System" time is the time for water to travel from the most remote point in the storm drainage system to the upstream end of the storm drain run under consideration. For the upstream most storm drain run, this value will be the same as the value in Column 9. For all other pipe runs, this value is computed by adding the "System" time of concentration (Column 10) and the "Section" time of concentration (Column 17) from the previous run together to get the system time of concentration at the upstream end of the section under consideration.

Step 4 Using the information from Step 3, compute the following:

a. "TOTAL" area, Column 5

Add the incremental area in Column 4 to the previous sections total area and place this value in Column 5.

b. "INC." area x "C," Column 7

Multiply the drainage area in Column 4 by the runoff coefficient in Column 6. Put the product, CA, in Column 7.

c. "TOTAL" area x "C," Column 8

Add the value in Column 7 to the value in Column 8 for the previous storm drain run and put this value in Column 8.

d. "I," Column 11

Using the larger of the two times of concentration in Columns 9 and 10, and an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity, I, and place this value in Column 11.

e. "TOTAL Q," Column 12

Calculate the discharge as the product of Columns 8 and 11. Place this value in Column 12.

f. "SLOPE," Column 21

Place the pipe slope value in Column 21. The pipe slope will be approximately the slope of the finished roadway. The slope can be modified as needed.

g. "PIPE DIA.," Column 13

Size the pipe using relationships and charts presented in section 7.8.4 to convey the discharge by varying the slope and pipe size as necessary. The storm drain should be sized as close as possible to a full gravity flow. Since most calculated sizes will not be available, a nominal size will be used. The designer will decide whether to go to the next larger size and have part full flow or whether to go to the next smaller size and have pressure flow.

h. "CAPACITY FULL," Column 14

Compute the full flow capacity of the selected pipe using Equation 7.4 and put this information in Column 14.

i. "VELOCITIES," Columns 15 and 16

Compute the full flow and design flow velocities (if different) in the conduit and place these values in Columns 15 and 16. If the pipe is flowing full, the velocities can be determined from $V = Q/A$, Equation 7.3, or Chart 7.1. If the pipe is not flowing full, the velocity can be determined from Figure 7.1 or the appropriate pipe flow chart.

j. "SECTION TIME," Column 17

Calculate the travel time in the pipe section by dividing the pipe length (Column 3) by the design flow velocity (Column 16). Place this value in Column 17.

k. "CROWN DROP," Column 20

Calculate an approximate crown drop at the structure to offset potential structure energy losses using Equation 7.7 introduced in Section 7.8.8. Place this value in Column 20.

l. "INVERT ELEV.," Columns 18 and 19

Compute the pipe inverts at the upper (U/S) and lower (D/S) ends of this section of pipe, including any pipe size changes that occurred along the section.

Step 5 Repeat steps 3 and 4 for all pipe runs to the storm drain outlet. Use equations and nomographs to accomplish the design effort.

Step 6 Check the design by calculating the energy grade line and hydraulic grade line.

See Appendix 7B for an example problem.

7.9 Energy Grade Line/Hydraulic Grade Line

The energy grade line (EGL) represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head. The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. To establish the EGL, it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses:

- Friction losses can be calculated using the Manning's Equation.
- Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. Various tables and calculations exist for developing the value of K depending on the structure being evaluated for loss.

Knowledge of the location of the EGL is critical to the understanding and estimating the location of the hydraulic grade line (HGL). The HGL is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. HGL is determined by subtracting the velocity head ($V^2/2g$) from the EGL (see Figure 7.4).

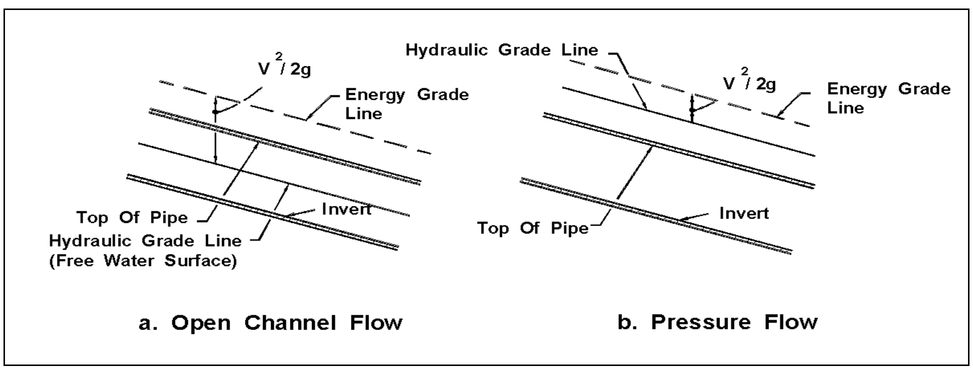


Figure 7.4. Hydraulic and energy grade lines in pipe flow.

In storm drains, there are three possible locations of the HGL:

- Open channel flow - When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface.
- Pressure flow - When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe and is the level to which water would rise in a vertical tube at any point along the pipe.
- Full gravity flow - When the flow in the pipe just reaches the point where the pipe is flowing full, this condition lies in between open channel flow and pressure flow. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

Inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. Storm drainage systems can often alternate between pressure and open channel flow conditions from one section to another.

7.9.1 Tailwater

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth of the outlet. The tailwater may also occur between the critical depth and the invert of the outlet. However, the starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

An exception to the above rule would be for a very large outfall with low tailwater where a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the design tailwater elevation, whichever was highest.

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 7.4 provides a comparison of discharge frequencies for coincidental occurrence for a 10- and 100-year design storm. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 2000 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 2000 to 2 which equals 1000 to 1. From Table 7.4, one would use the 2-year event on the stream draining the large watershed when establishing the tailwater for the use in analyzing a 10-year event for the small watershed. One would not expect the same event to occur for both size watersheds at the same time.

There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and access holes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition.

Table 7.4. Frequencies for Coincidental Occurrence.

Area Ratio	Frequencies for Coincidental Occurrence			
	10-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

The **orientation of the outfall** is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure cannot be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

7.9.2 Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. The following sections present relationships for estimating typical energy losses in storm drainage systems.

Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion at the outlet, the exit loss is:

$$H_o = C_o \left[\frac{V_O^{\,2}}{2g} - \frac{V_d^2}{2g} \right] \tag{7.9}$$

where:

V_O	=	Average outlet velocity, ft/s
v_d	=	Channel velocity downstream of outlet, ft/s
C_o	=	Exit loss coefficient (1.0)

Note that, when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For partial full flow where a properly aligned pipe outlets into a channel with moving water, the exit loss may be reduced to virtually zero.

Pipe Friction Losses

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows

$$H_f = S_f L \tag{7.10}$$

where:

H_f = Friction loss, ft
S_f = Friction slope, ft/ft
L = Length of pipe, ft

The friction slope in Equation 7.10 is also the slope of the hydraulic gradient for a particular pipe run. As indicated by Equation 7.10, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow can be determined by combining Equation 7.10 with the Manning's equation as follows:

$$S_f = \left(\frac{Q\ n}{1.486\ A R^{2/3}} \right)^2 \tag{7.11}$$

Equation 7.11 is for any shape of conduit. For a circular pipe flowing full, the following form may be developed:

$$S_f = \left(\frac{Q\ n}{0.463\ D^{2.67}} \right)^2 \tag{7.12}$$

Combining Equations 7.10 and 7.12, the following form is found:

$$H_f = 4.665 \left(\frac{Q\ n}{D^{2.67}} \right)^2 L \tag{7.13}$$

Bend Loss

The bend loss coefficient for storm drain design is minor but can be evaluated using the formula:

$$h_b = 0.0033\ (\Delta)(V_o^2 / 2g) \tag{7.14}$$

where:

Δ = Angle of curvature, degrees

Junction Losses

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. The minor loss equation for a pipe junction is a form of the momentum equation as follows:

$$H_J = \frac{(Q_0\, V_0) - (Q_L\, V_i) - (Q_i\, V_L\, \mathrm{Cos}\,\theta)}{0.5\, g\, (A_0 + A_i)} + h_i - h_o \qquad (7.15)$$

where:

H_J = Junction loss, ft
Q_o, Q_i, Q_L = Outlet, inlet, and lateral flows respectively, ft^3/s
V_o, V_i, V_L = Outlet, inlet, and lateral velocities, respectively, ft/s
h_o, h_i = Outlet and inlet velocity heads, ft
A_o, A_i = Outlet and inlet cross-sectional areas, ft^2
θ = Angle between the inflow and outflow pipes (Figure 7.2)

Access Hole and Inlet Losses

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by Equation 7.16. Experimental studies have determined that the K value can be approximated by the relationship in Equation 7.17 when the inflow pipe invert is below the water level in the access hole.

$$H_{ah} = K\left(\frac{V_o^2}{2g}\right) \qquad (7.16)$$

where:

$$K = K_o C_D C_d C_Q C_p C_B \qquad (7.17)$$

where:

K = Adjusted loss coefficient
K_o = Initial head loss coefficient based on relative access hole size
C_D = Correction factor for pipe diameter (pressure flow only)
C_d = Correction factor for flow depth (non-pressure flow only)
C_Q = Correction factor for relative flow
C_p = Correction factor for plunging flow
C_B = Correction factor for benching

For cases where the inflow pipe invert is above the access hole water level, the outflow pipe will function as a culvert, and the access hole loss and the access hole HGL can be computed using procedures found in *Hydraulic Design of Highway Culverts* (HDS-5).[7] If the outflow pipe is flowing full or partially full under outlet control, the access hole loss (due to flow contraction into the outflow pipe) can be computed by setting K in Equation 7.16 to K_e as reported in Table 7.5. If the outflow pipe is flowing under inlet control, the water depth in the access hole should be computed using the inlet control nomographs in HDS-5[7] (for example see Charts 7.3 and 7.4 in Appendix B).

Table 7.5. Entrance Loss Coefficients.[7]

Type of Structure and Design of Entrance	Coefficient K_e
• **Pipe, Concrete**	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (grove end)	0.2
Square-edge	0.5
Rounded (radius – D/12)	0.2
Mitered to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
• **Pipe, or Pipe-Arch, Corrugated Metal**	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45 ^{0} bevels	0.2
Side- or slope-tapered inlet	0.2
• **Box, Reinforced Concrete**	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30^0 to 75^0 to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10^0 to 25^0 to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Note: "End sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both <u>inlet</u> and <u>outlet</u> control. Some end sections, incorporating a <u>closed</u> taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Relative Access Hole Size

K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes (see Figure 7.2):

$$K_o = 0.1(b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta \quad (7.18)$$

where:

θ	=	Angle between inflow and outflow pipes, degrees
b	=	Access hole diameter
D_o	=	Outlet pipe diameter

Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure-flow situations where the depth in the access hole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, it is only applied in such cases:

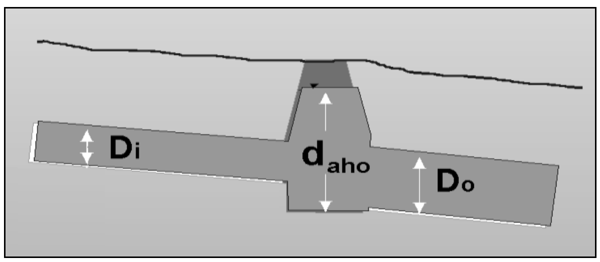


Figure 7.5. Pipe diameter.

$$C_D = (D_o / D_i)^3 \quad (7.19)$$

where:

D_i	=	Incoming pipe diameter
D_o	=	Outgoing pipe diameter

Flow Depth

The correction factor for flow depth is significant only in free surface flow or low pressures, where the d/D_o ratio is less than 3.2, and is only applied in such cases. Water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

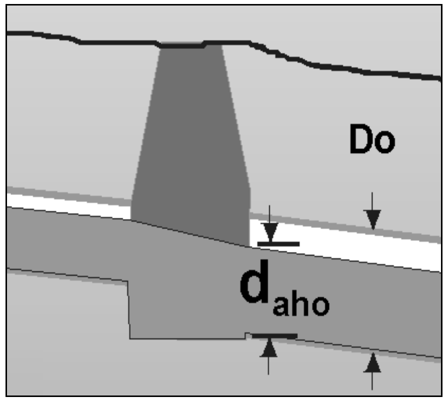


Figure 7.6. Flow depth.

$$C_d = 0.5\left(\frac{d_{aho}}{D_0}\right)^{0.6} \tag{7.20}$$

where:

d	=	Water depth in access hole above outlet pipe, ft
D_o	=	Outlet pipe diameter, ft

Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \tag{7.21}$$

where:

C_Q	=	Correction factor for relative flow
θ	=	Angle between the inflow and outflow pipes, degrees
Q_i	=	Flow in the inflow pipe, ft^3/s
Q_o	=	Flow in the outlet pipe, ft^3/s

As can be seen from the Equation, C_Q is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access hole shown in Figure 7.7 and assume the following two cases to determine the impact of Pipe 2 entering the access hole:

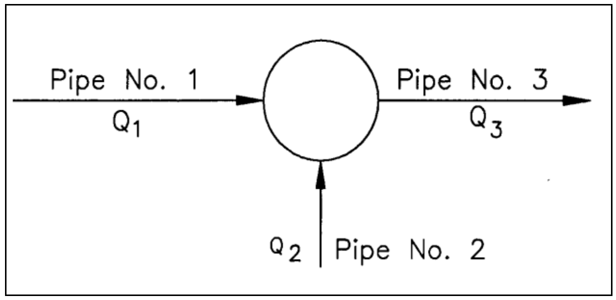


Figure 7.7. Relative flow.

<u>Case 1</u>

$Q_1 = 3.2\ ft^3/s$, $Q_2 = 1.0\ ft^3/s$, $Q_3 = 4.2\ ft^3/s$
$C_{Q3\text{-}1} = (1 - 2\sin 180°)(1 - 3.2/4.2)^{0.75} + 1 = 1.34$

<u>Case 2</u>

$Q_1 = 1.0\ ft^3/s$, $Q_2 = 3.2\ ft^3/s$, $Q_3 = 4.2\ ft^3/s$
$C_{Q2\text{-}1} = (1 - 2\sin 90°)(1 - 3.2/4.2)^{0.75} + 1 = 0.66$

Plunging Flow

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 7.8 for the Example, C_p is calculated for Pipe # 1 when Pipe # 2 discharges plunging flow. The correction factor is only applied when h > d. The correction factor for plunging flow, C_p, is calculated by the following:

$$C_p = 1 + 0.2\left[\frac{h}{D_o}\right]\left[\frac{(h - d_{aho})}{D_o}\right] \qquad (7.22)$$

where:

C_p	=	Correction for plunging flow
h	=	Vertical distance from flow line of incoming pipe to center of outlet pipe, ft
D_o	=	Outlet pipe diameter, ft
d_{aho}	=	Water depth in access hole relative to outlet pipe invert as shown in Figure 7.8, ft

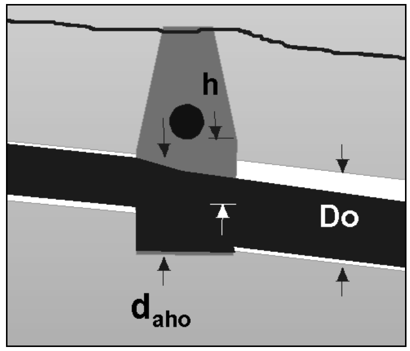


Figure 7.8. Plunging flow.

Benching

The correction for benching in the access hole, C_B, is obtained from Table 7.6. Benching tends to direct flows through the access hole, resulting in reductions in head loss (Figures 7.9 and 7.10). For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed. Benching should only be used where energy losses must be kept to a minimum. In areas where energy is not a problem, there is no need to use benching.

Table 7.6. Corrections for Benching.

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat or depressed floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02

*pressure flow, d/D_o>3.2 **free surface flow, d/D_o<1.0

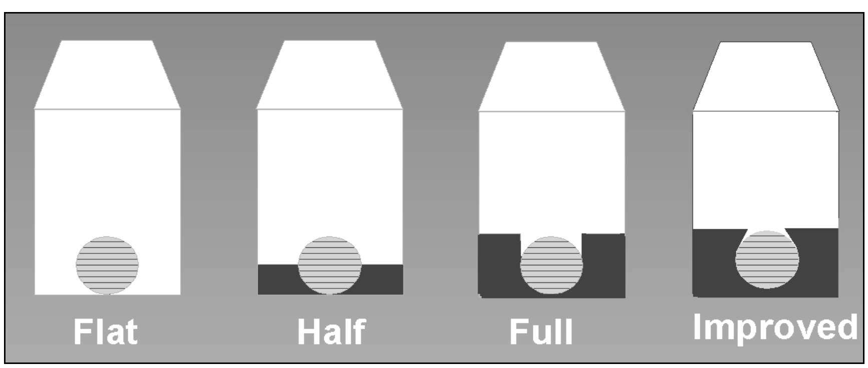


Figure 7.9. Types of benches.

Figure 7.10. Example of bench in access hole.

Other Energy Losses

There are other types of energy losses that may be part of the storm drain system, but are not covered here and should be evaluated when present. These losses may be caused by transitions due to expansions and contractions or obstructions. For information on how to handle these losses see FHWA's Hydraulic Engineering Circular Number 22.[1]

7.9.3 Summary

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

7.10 Energy Grade Line Evaluation Procedure

This section presents a step-by-step procedure for manual calculation of the energy grade line (EGL) and the hydraulic grade line (HGL). For most storm drainage systems, computer methods such as HYDRA[8] are the most efficient means of evaluating the EGL and the HGL. However, it is important that the designer understand the analysis process so that he can better interpret the output from computer generated storm drain designs.

Figure 7.11 provides a sketch illustrating use of the two grade lines in developing a storm drainage system. The following step-by-step procedure can be used to manually compute the EGL and HGL. The computation tables in Figures 7.12 and 7.13 can be used to document the procedure outlined below.

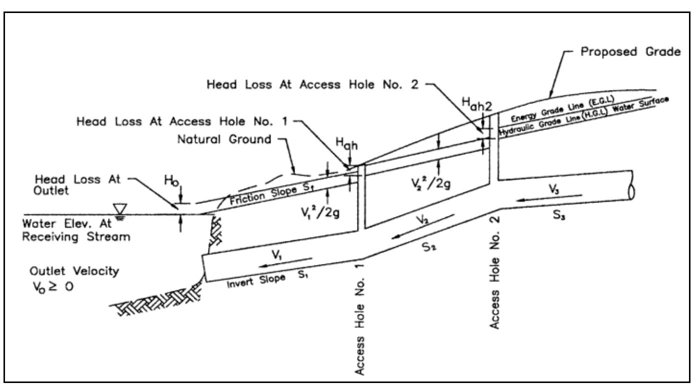


Figure 7.11. Energy and hydraulic grade line illustration.

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______
INITIAL TAILWATER ELEV. ______

Str. ID	D	Q	L	V	d	d_c	$V^2/2g$	S_f	Total Pipe Loss (table B)	EGL_o	K (table B)	$K(V^2/2g)$	EGL_i	HGL	U/S TOC	Surf. Elev.
	( )	(^{3}/s)	( )	(/s)	( )	( )	( )	(/)	( )	( )		( )	( )	( )	( )	( )
(1)	(2)	(3)	(4)	(5)	(6a)	(6b)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)

Figure 7.12. Energy grade line computation sheet - Table A.

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B

COMPUTED BY ________ DATE ________ ROUTE ____________________
CHECKED BY ________ DATE ________ SECTION ____________________
PAGE ________ OF ________ COUNTY ____________________

	Pipe Losses ( )						Structure Losses ( )							
Str. ID (1)	H_f (2)	h_b (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)

Figure 7.13. Energy grade line computation sheet - Table B.

Before outlining the computational steps in the procedure, a comment relative to the organization of data on the form is appropriate. In general, a line will contain the information on a specific structure and the line downstream from the structure. As the table is started, the first two lines may be unique. The first line will contain information about the outlet conditions. This may be a pool elevation or information on a known downstream system. The second line will be used to define the conditions right at the end of the last conduit. Following these first two lines the procedure becomes more general. A single line on the computation sheet is used for each junction or structure and its associated outlet pipe. For example, data for the first structure immediately upstream of the outflow pipe and the outflow pipe would be tabulated in the third full line of the computation sheet (lines may be skipped on the form for clarity). Table A (Figure 7.12) is used to calculate the HGL and EGL elevations while table B (Figure 7.13) is used to calculate the pipe losses and structure losses. Values obtained in table B are transferred to table A for use during the design procedure. In the description of the computation procedures, a column number will be followed by a letter A or B to indicate the appropriate table to be used.

EGL computations begin at the outfall and are worked upstream taking each junction into consideration. Many storm drain systems are designed to function in a subcritical flow regime. In subcritical flow or full barrel flow, pipe and access hole losses are summed to determine the upstream EGL levels. **If supercritical flow occurs, pipe and access losses are not carried upstream**. When a storm drain section is identified as being supercritical, the designer should advance to the next upstream pipe section to determine its flow regime. This process continues until the storm drain system returns to a subcritical flow regime.

The EGL computational procedure follows:

Step 1 The first line of Table A includes information on the system beyond the end of the conduit system. Define this as the stream, pool, existing system, etc. in column 1A. Determine the EGL and HGL for the downstream receiving system. If this is a natural body of water, the HGL will be at the water surface. The EGL will also be at the water surface if no velocity is assumed or will be a velocity head above the HGL if there is a velocity in the water body. If the new system is being connected to an existing storm drain system, the EGL and the HGL will be that of the receiving system. Enter the HGL in Column 14A and the EGL in Column 10A of the first line on the computation sheet.

Step 2 Identify the structure number at the outfall (this may be just the end of the conduit, but it needs a structure number), the top of conduit (TOC) elevation at the outfall end, and the surface elevation at the outfall end of the conduit. Place these values in Columns 1A, 15A, and 16A respectively. Also add the structure number in Col.1B.

Step 3 Determine the EGL just upstream of the structure identified in Step 2. Several different cases exist as defined below when the conduit is flowing full:

Case 1: If the TW at the conduit outlet is greater than $(d_c + D)/2$, the EGL will be the TW elevation plus the velocity head for the conduit flow conditions.

Case 2: If the TW at the conduit outlet is less than $(d_c + D)/2$, the EGL will be the HGL plus the velocity head for the conduit flow conditions. The equivalent hydraulic grade line, EHGL, will be the invert plus $(d_c + D)/2$.

The velocity head needed in either Case 1 or 2 will be calculated in the next steps, so it may be helpful to complete Step 4 and work Step 5 to the point where velocity head (Col. 7A) is determined and then come back and finish this step. Put the EGL in Column 13A.

Note: The values for d_c for circular pipes can be determined from Chart 7.2. Charts for other conduits or other geometric shapes can be found in *Hydraulic Design of Highway Culverts*, HDS-5, and cannot be greater than the height of the conduit.

Step 4 Identify the structure ID for the junction immediately upstream of the outflow conduit (for the first conduit) or immediately upstream of the last structure (if working with subsequent lines) and enter this value in Columns 1A and 1B of the next line on the computation sheets. Enter the conduit diameter (D) in column 2A, the design discharge (Q) in Column 3A, and the conduit length (L) in Column 4A.

Step 5 If the barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in column 7A. Put “full” in Column 6a and not applicable (n/a) in Column 6b of Table A. Continue with Step 6. If the barrel flows only partially full, continue with Step 5A.

Note: If the pipe is flowing full because of high tailwater or because the pipe has reached its capacity for the existing conditions, the velocity will be computed based on continuity using the design flow and the full cross sectional area. Do not use the full flow velocity determined in Column 15 of the Preliminary Storm Drain Computation Form for part-full flow conditions. For part-full conditions discussed in Step 5, the calculations in the preliminary form may be helpful. Actual flow velocities need to be used in the EGL/HGL calculations.

Step 5A Part full flow: Using the hydraulic elements graph in Chart 7.5 in Appendix 7-B with the ratio of part full to full flow (values from the preliminary storm drain computation form), compute the depth and velocity of flow in the conduit. Enter these values in Column 6a and 5 respectively of Table A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

Step 5B Compute critical depth for the conduit using Chart 7.2 in Appendix 7-B. If the conduit is not circular, see HDS-5(2) for additional charts. Enter this value in Column 6b of Table A.

Step 5C Compare the flow depth in Column 6a (Table A) with the critical depth in Column 6b (Table A) to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 6. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5D. In either case, remember that the EGL must be higher upstream for flow to occur. If after checking for super critical flow in the upstream section of pipe, assure that the EGL is higher in the pipe than in the structure.

Step 5D Pipe losses in a supercritical pipe section are not carried upstream. Therefore, enter a zero (0) in Column 7B for this structure.

Step 5E Enter the structure ID for the next upstream structure on the next line in Columns 1A and 1B. Enter the pipe diameter (D), discharge (Q), and conduit length (L) in Columns 2A, 3A, and 4A respectively of the same line.

Note: After a downstream pipe has been determined to flow in supercritical flow, it is necessary to check each succeeding upstream pipe for the type of flow that exists. This is done by calculating normal depth and critical depth for each pipe. If normal depth is less than the diameter of the pipe, the flow will be open channel flow and the critical depth calculation can be used to determine whether the flow is sub or supercritical. If the flow line elevation through an access hole drops enough that the invert of the upstream pipe is not inundated by the flow in the downstream pipe, the designer goes back to Step1A and begins a new design as if the downstream section did not exist.

Step 5F Compute normal depth for the conduit using Chart 7.5 and critical depth using Chart 7.2. If the conduit is not circular see HDS-5[(7)] for additional charts. Enter these values in Columns 6A and 6b of Table A.

Step 5G If the pipe barrel flows full, enter the full flow velocity from continuity in Column 5A and the velocity head ($V^2/2g$) in Column 7A. Go to Step 3, Case 2 to determine the EGL at the outlet end of the pipe. Put this value in Column 10A and go to Step 6. For part full flow, continue with Step 5H.

Step 5H Part full flow: Compute the velocity of flow in the conduit and enter this value in Column 5A. Compute the velocity head ($V^2/2g$) and place in Column 7A.

Step 5I Compare the flow depth in Column 6a with the critical depth in Column 6b to determine the flow state in the conduit. If the flow depth in Column 6a is greater than the critical depth in Column 6b, the flow is subcritical, continue with Step 5J. If the flow depth in Column 6a is less than or equal to the critical depth in Column 6b, the flow is supercritical, continue with Step 5K.

Step 5J Subcritical flow upstream: Compute EGL_o at the outlet of the previous structure as the outlet invert plus the sum of the outlet pipe flow depth and the velocity head. Place this value in Column 10A of the appropriate structure and go to Step 9.

Step 5K Supercritical flow upstream: Access hole losses do not apply when the flow in two (2) successive pipes is supercritical. Place zeros (0) in Columns 11A, 12A, and 15B of the intermediate structure (previous line). The HGL at the structure is equal to the pipe invert elevation plus the flow depth. Check the invert elevations and the flow depths both upstream and downstream of the structure to determine where the highest HGL exists. The highest value should be placed in Column 14A of the previous structure line. Perform Steps 20 and 21 and then repeat Steps 5E through 5K until the flow regime returns to subcritical. If the next upstream structure is end-of-line, skip to step 10b then perform Steps 20, 21, and 24.

Step 6 Compute the friction slope (S_f) for the pipe using Equation 7.12:

$$S_f = H_f / L = [Q n / (0.463 D^{2.67})]^2$$

Enter this value in Column 8A of the current line. Equation 7.12 assumes full flow in the outlet pipe. If full flow does not exist, set the friction slope equal to the pipe slope.

Step 7 Compute the friction loss (H_f) by multiplying the length (L) in Column 4A by the friction slope (S_f) in Column 8A and enter this value in Column 2B. Compute other losses along the pipe run such as bend losses (h_b), transition contraction (H_c) and expansion (H_e) losses, and junction losses (H_j) and place the values in Columns 3B, 4B, 5B, and 6B, respectively. Add the values in 2B, 3B, 4B, 5B, and 6B and place the total in Column 7B and 9A.

Step 8 Compute the energy grade line value at the outlet of the structure (EGL_o) as the EGL_i elevation from the previous structure (Column 13A) plus the total pipe losses (Column 9A). Enter the EGL_o in Column 10A.

Step 9 Estimate the depth of water in the access hole (estimated as the depth from the outlet pipe invert to the hydraulic grade line in the pipe at the outlet). Computed as EGL_o (Column 10A) minus the pipe velocity head in Column 7A minus the pipe invert elevation (from the preliminary storm drain computation form). Enter this value in Column 8B. If supercritical flow exists in this structure, leave this value blank and skip to Step 5E.

Step 10 If the inflow storm drain invert is submerged by the water level in the access hole, compute access hole losses using Equations 7.16 and 7.17. Start by computing the initial structure head loss coefficient, K_o, based on relative access hole size. Enter this value in Column 9B. Continue with Step 11. If the inflow storm drain invert is not submerged by the water level in the access hole, compute the head in the access hole using culvert techniques from HDS-5[(7)] as follows:

a. If the structure outflow pipe is flowing full or partially full under outlet control, compute the access hole loss by setting K in Equation 7.16 to K_e as reported in Table 7.5. Enter this value in Column 15B and 11A, continue with Step 17. Add a note on Table A indicating that this is a drop structure.

b. If the outflow pipe functions under inlet control, compute the depth in the access hole (HGL) using Chart 7.3 or 7.4. If the storm conduit shape is other than circular, select the appropriate inlet control nomograph from HDS-5.[(2)] Add these values to the access hole invert to determine the HGL. Since the velocity in the access hole is negligible, the EGL and HGL are the same. Enter HGL in Col.14A and EGL in Col.13A. Add a note on Table A indicating that this is a drop structure. Go to Step 20.

Step 11 Using Equation 7.19 compute the correction factor for pipe diameter, C_D, and enter this value in Column 10B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is greater than 3.2.

Step 12 Using Equation 7.20 compute the correction factor for flow depth, C_d, and enter this value in Column 11B. Note, this factor is only significant in cases where the d_{aho}/D_o ratio is less than 3.2.

Step 13 Using Equation 7.21, compute the correction factor for relative flow, C_Q, and enter this value in Column 12B. This factor = 1.0 if there are less than 3 pipes at the structure.

Step 14 Using Equation 7.22, compute the correction factor for plunging flow, C_p, and enter this value in Column 13B. This factor = 1.0 if there is no plunging flow. This correction factor is only applied when $h>d_{aho}$.

Step 15 Enter in Column 14B the correction factor for benching, C_B, as determined from table 7.6. Linear interpolation between the two columns of values will most likely be necessary.

Step 16 Using Equation 7.17, compute the value of K and enter this value in Column 15B and 11A.

Step 17 Compute the total access hole loss, H_{ah}, by multiplying the K value in Column 11A by the velocity head in Column 7A. Enter this value in Column 12A.

Step 18 Compute EGL_i at the structure by adding the structure losses in Column 12A to the EGL_o value in Column 10A. Enter this value in Column 13A.

Step 19 Compute the hydraulic grade line (HGL) at the structure by subtracting the velocity head in Column 7A from the EGL_i value in Column 13A. Enter this value in Column 14A.

Step 20 Determine the top of conduit (TOC) value for the inflow pipe (using information from the storm drain computation sheet) and enter this value in Column 15A.

Step 21 Enter the ground surface, top of grate elevation or other high water limits at the structure in Column 16A. If the HGL value in Column 14A exceeds the limiting elevation, design modifications will be required.

Step 22 Enter the structure ID for the next upstream structure in Column 1A and 1B of the next line. When starting a new branch line, skip to Step 24.

Step 23 Continue to determine the EGL through the system by repeating Steps 4 through 23. (Begin with Step 2 if working with a drop structure. This begins the design process again as if there were no system down stream from the drop structure).

Step 24 When starting a new branch line, enter the structure ID for the branch structure in Column 1A and 1B of a new line. Transfer the values from Columns 2A through 10A and 2B to 7B associated with this structure on the main branch run to the corresponding columns for the branch line. If flow in the main storm drain at the branch point is subcritical, continue with Step 9; if supercritical, continue with Step 5E.

7.11 Computer Programs

A variety of computer programs are available to facilitate storm drain design. The use of any of these programs is acceptable, provided the program substantially conforms to the theory and methods described in HEC-22.

7.12 Additional Guidance

- Flap Gates. When necessary, backflow protection should be provided in the form of flap gates. These gates offer negligible resistance to the release of water from the system and their effect upon the hydraulics of the system may be neglected.

- Perforated underdrain pipe should be placed in areas where a subsurface permeable layer is needed. See Ga Standard 9029B. Example locations include under curb and gutter sections at the low side of superelevation and at low points of sag vertical curves on tangent sections.

- Pipe charts in Appendix A Select the chart by assuming a pipe size. Check to see if the chart selected is appropriate for your design criteria.

- Generally, a storm drain should be kept as close to the surface as minimum cover and/or hydraulic requirements will allow to minimize excavation costs.

- Tip: Coordinate with utility locations. Gravity systems such as sanitary sewers should be closely checked for conflicts. Pressure fed systems like water and gas can usually be routed to avoid the gravity flow systems.

7.13 Represent Drainage Design on the Plans

- Drainage information is to be represented on the plans, drainage profiles, quantities as per the plan presentation guide.

- Include in the plans the pipe selection table from the soils report. This is typically shown on the drainage quantity sheet.

References

1. Brown, S.A., Stein, S.M., and Warner, J.C., 2001, *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22, FHWA-NHI-01-021. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

2. AASHTO, *A Policy on Geometric Design of Highways and Streets*, Task Force on Geometric Design, 2001.

3. American Association of State Highway and Transportation Officials, 1991. *Model Drainage Manual*. “Chapter 13: Storm Drainage Systems,” AASHTO, Washington, D.C.

4. AASHTO, *Highway Drainage Guidelines*, Chapter 9, “Storm Drain Systems,” Task Force on Hydrology and Hydraulics, 2003.

5. Federal Highway Administration, *Bridge Deck Drainage Systems*, Hydraulic Engineering Circular No. 21, FHWA-SA-92-010, 1993.

6. Federal Highway Administration, Hydraulic Design Series No. 3, *Design Charts for Open-Channel Flow*, FHWA-EPD-86-102, 1961.

7. Normann, J.M., Houghtalen, R.J., and Johnston, W.J., 2001. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5, Federal Highway Administration, FHWA-NHI-01-020, Washington, D.C.

8. GKY & Associates, 1994. HYDRAIN -Integrated Drainage Design Computer System; Version 5.0. "Volume III: HYDRA - Storm Drains," FHWA-RD-92-061, U.S. Department of Transportation, Federal Highway Administration, McLean, VA.

9. American Concrete Pipe Association, "Concrete Pipe Design Manual," American Concrete Pipe Association, Irving, TX, 2000.

APPENDIX - CHAPTER 7

Appendix A – Example Problems
Appendix B – Design Charts

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7.14 APPENDIX A – Example Problems

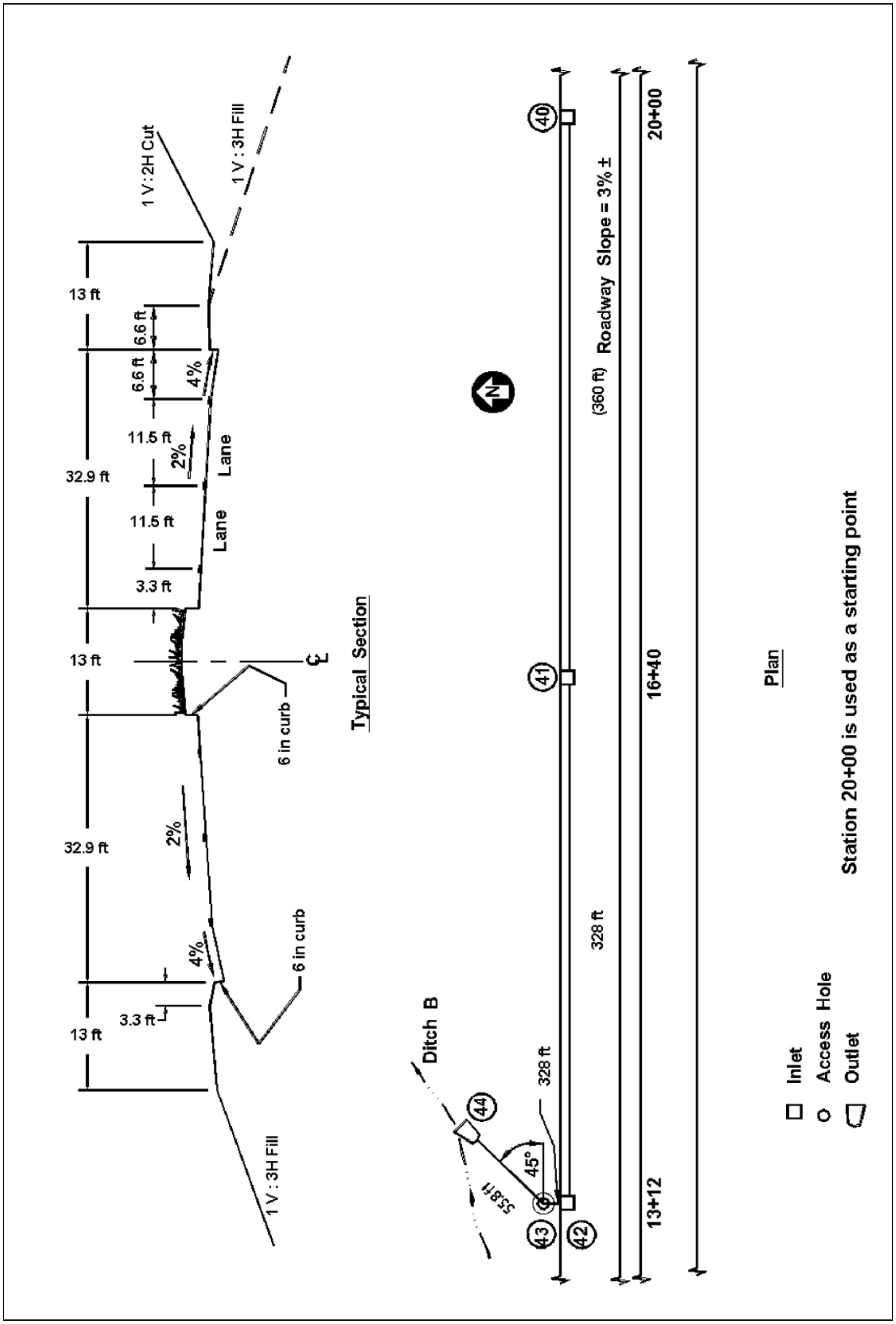


Figure 7A.1. Plan view of example problem sketch.

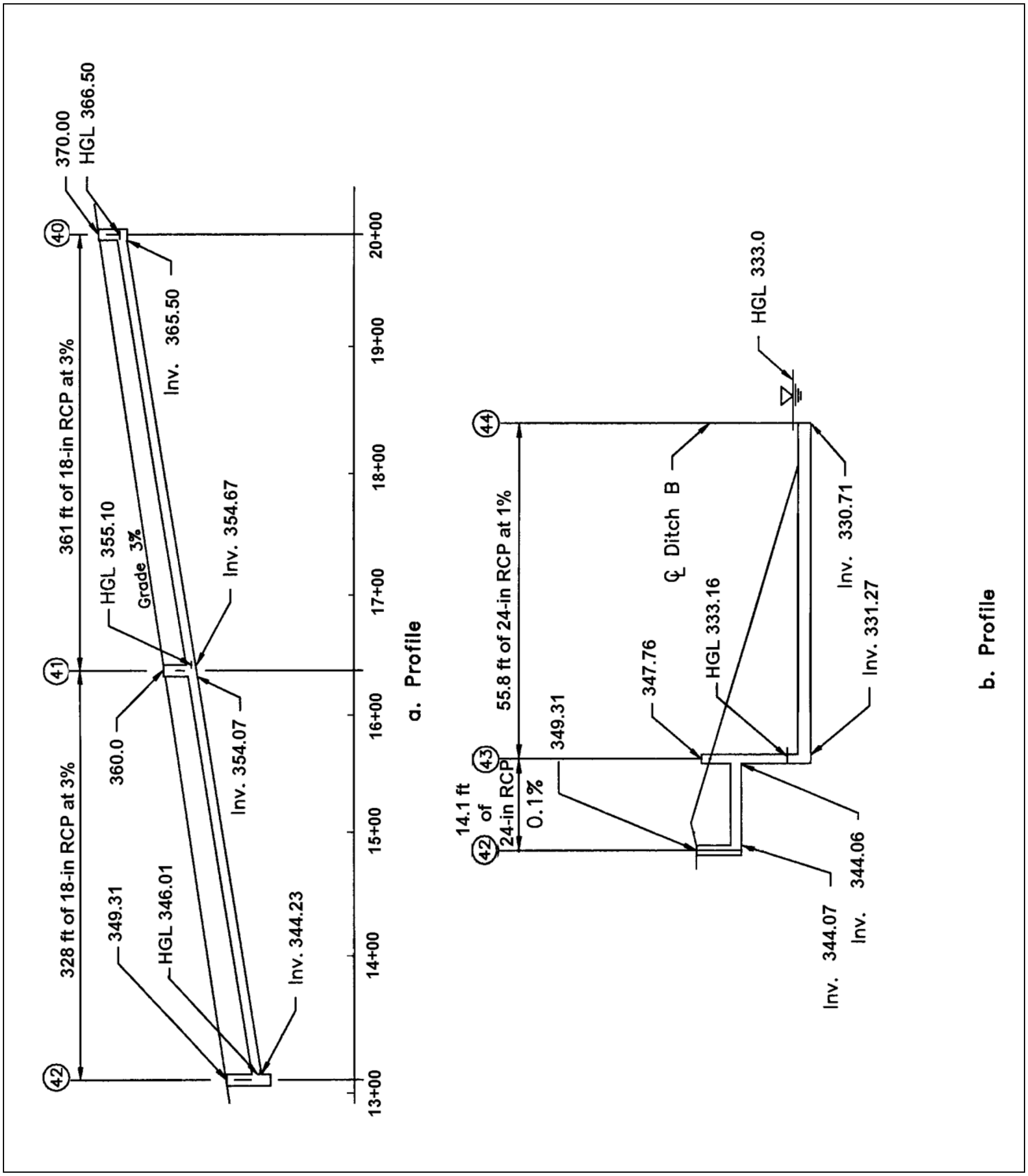


Figure 7A.2. Profile of storm drain example problem.

Storm Drain Design Examples

The following storm drain design examples illustrate the application of the design procedures as outlined in this chapter.

Example Problem 7.1 Preliminary Storm Drain Design

Given: *The roadway plan and section illustrated in figure 7A.1, duration intensity information in table 7A.2, and inlet drainage area information in table 7A.1 are provided. All grates are type P 50 x 100, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter = 18 in for maintenance purposes.*

Find:

(1) Using the procedures outlined in Section 7.8.9 determine appropriate pipe sizes and inverts for the system illustrated in Figure 7A.1.

(2) Evaluate the HGL for the system configuration determined in part (1) using the procedure outlined in Section 7.10.

Solution:

(1) Preliminary Storm Drain Design

Step 1. Figure 7A.1 illustrates the proposed system layout including location of storm drains, access holes, and other structures. All structures have been numbered for reference. Figure 7A.2 (a) and (b) illustrate the corresponding storm drain profiles.

Step 2. Drainage areas, runoff coefficients, and times of concentration are tabulated in Table 7A.1. Example problems documenting the computation of these values are included in chapter 6.

Starting at the upstream end of a conduit run, Steps 3 and 4 from Section 7.8.9 are completed for each storm drain pipe. A summary tabulation of the computational process is provided in Figure 7A.3. The column-by-column computations for each section of conduit follow:

Table 7A.1. Drainage Area Information for Design Example.			
Inlet No.	Drainage Area (ac)	C	Time of Concentration (min)
40	0.64	0.73	3
41	0.35	0.73	2
42	0.32	0.73	2
43	--	--	--
44			

Table 7A.2. Intensity/Duration Data for Example Problem.									
Time (min)	5	10	15	20	30	40	50	60	120
Intensity (in/hr)	7.1	5.9	5.1	4.5	3.5	3.0	2.6	2.4	1.4

PRELIMINARY STORM DRAIN COMPUTATION SHEET
(English Solution)

COMPUTED BY ______ DATE ______ ROUTE __________
CHECKED BY ______ DATE ______ SECTION __________
PAGE ______ OF ______ COUNTY __________

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	(Ft)	INC (ac)	TOTAL (ac)		INC. (ac)	TOTAL (ac)	INLET (min)	SYSTEM (min)	(in/hr)	(ft^3 /s)	(ft)	(ft^3/s)	FULL (ft /s)	DESIGN (ft /s)	(min)	U/S (ft)	D/S (ft)	(ft)	(ft/ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
40	41	361.0	0.64	0.64	0.73	0.47	0.47	3	5	7.1	3.3	1.50	18.1	10.3	7.52	1	365.50	354.67	----	0.03
41	42	328.0	0.35	0.99	0.73	0.25	0.72	2	5	7.1	5.1	1.5	18.1	10.3	8.7	1	354.07	344.23	0.6	0.03
42	43	14.1	0.32	1.31	0.73	0.23	0.95	2	5	7.1	6.75	2.0	7.12	2.28	2.6	0	344.07	344.06	0.16	0.001
43	44	55.8	0.00	1.31	-	-	0.95	-	5	7.1	6.75	2.0	22.52	7.22	6.10	0	331.27	330.71	12.79	0.01

Figure 7A.3. Storm drain computation sheet.

<u>Structure 40 to 41</u>

Col. 1 From structure 40		
Col. 2 To structure 41		
Col. 3 Run Length	$L = 2000$ ft - 1639 ft $L = 361$ ft	*figure 7A.2*
Col. 4 Inlet Area	$A_i = 0.64$ ac	*table 7A.1*
Col. 5 Total Area	$A_t = 0.64$ ac	*total area up to inlet 40*
Col. 6 "C"	$C = 0.73$	*table 7A.1*
Col. 7 Inlet CA	$CA = (0.64)(0.73)$ $CA = 0.47$ ac	*Col. 4 times Col. 6*
Col. 8 Sum CA	*ƷCA = 0.47 + 0* *ƷCA= 0.47 ac*	*Col. 7 plus previous Col. 8*
Col. 9 Inlet Time	$t_i = 3$ min	*table 7A.1*
Col. 10 Sys. Time	$t_c = 3$ min (use 5 min)	*same as Col. 9 for upstream most section*
Col. 11 Intensity	$I = 7.1$ in/hr	*table 7A.2 System time less than 5 minutes therefore, use 5 minutes*
Col. 12 Runoff	$Q = (CA)\ (I)\ /K_u$ $Q = (0.47)\ (7.1)\ /\ 1.0$ $Q = 3.3\ ft^3/sec$	*Rational equation* $K_u = 1.0$ *Col. 8 times Col. 11 divided by 1.0*
Col. 21 Slope	$S = 0.03$	*select desired pipe slope*
Col. 13 Pipe Dia.	$D = [(Qn)/(0.463\ S_o^{0.5})]^{0.375}$ $D = [(3.3)(0.013)/(0.46)(0.03)^{0.5}]^{0.375}$ $D = 0.8$ ft $D_{min} = 1.5$ ft	*equation 7.5 or chart 7.1* *use* D_{min}
Col. 14 Full Cap.	$Q_f = (0.463/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)\ (1.5)^{2.67}\ (0.03)^{0.5}$ $Q_f = 18.1 ft^3/s$	*equation 7.4 or chart 7.1*
Col. 15 Vel. Full	$V_f = (0.590/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)\ (1.5)^{0.67}\ (0.03)^{0.5}$ $V_f = 10.3$ ft/s	*equation 7.1 or chart 7.1*

Col. 16 Vel. Design	*$Q/Q_f = 3.3/18.1 = 0.18$* *$V/V_f = 0.73$* *$V = (0.73)\ (10.3)$* *$V = 7.52$ ft/s*	 *chart 7.5*
Col. 17 Sect. Time	*$t_s = L/V = 361 / 7.52 / 60$* *$t_s = 0.8$ min; use 1 min*	*Col. 3 divided by Col. 16*
Col. 20 Crown Drop	*= 0*	*Upstream most invert*
Col. 18 U/S Invert	*= Grnd - 3.0 ft - dia* *= 370.0 - 3.0 - 1.5* *= 365.5 ft*	*3 ft = min cover* *Ground elevation from figure 7A.2*
Col. 19 D/S Invert	*= (365.5) - (361.0)(0.03)* *= 354.67 ft*	*Col. 18 - (Col. 3)(Col. 21)*

At this point, the pipe should be checked to determine if it still has adequate cover.

354.67 + 1.5 + 3.0 = 359.17 *Invert elev. + Diam + min cover*
Ground elevation of 360.0 ft is greater than 359.17 ft so OK

<u>*Structure 41 to 42*</u>

Col. 1 From	*= 41*	
Col. 2 To	*= 42*	
Col. 3 Run Length	*L = 1639 - 1311* *L = 328 ft*	*figure 7A.2*
Col. 4 Inlet Area	*$A_i = 0.35$ ac*	*table 7A.1 or example 4-15*
Col. 5 Total Area	*$A_t = 0.35 + 0.64$* *$A_t = 0.99$ ac*	
Col. 6 "C"	*C = 0.73*	*table 7A.1*
Col. 7 Inlet CA	*CA = (0.35)(0.73)* *CA = 0.25 ac*	*Col. 4 times Col. 6*
Col. 8 Sum CA	*3CA = 0.25 + 0.47* *3CA = 0.72 ac*	*Col. 7 plus previous Col. 8*
Col. 9 Inlet Time	*$t_i = 2$ min*	*table 7A.1 (example 4-15)*

Col. 10 Sys. Time	$t_c = 4$ *min (use 5 min)*	*Col. 9 + Col. 17 for line 40-41*
Col. 11 Intensity	$I = 7.1$ *in/hr*	*table 7A.2; system time equals 5 min*
Col. 12 Runoff	$Q = (CA)(I)/(K_u)$ $Q = (0.72)\ (7.1)\ /\ 1.0$ $Q = 5.1\ ft^3/sec$	 *Col. 8 times Col. 11 divided by 1.0*
Col. 21 Slope	$S = 0.03$	*Select desired pipe slope*
Col. 13 Pipe Dia.	$D = [(Qn)/(0.463\ S_o^{0.5})]^{0.375}$ $D = [(5.1)\ (0.013)/(0.46)(0.03)^{0.5}]^{0.375}$ $D = 0.93\ ft$ $D_{min} = 1.5\ ft$	*equation 7.5 or chart 7.1* *use* D_{min}
Col. 14 Full Cap.	$Q_f = (0.463/n)\ D^{2.67}\ S_o^{0.5}$ $Q_f = (0.46/0.013)(1.5)^{2.67}(0.03)^{0.5}$ $Q_f = 18.1\ ft^3/s$	*equation 7.4 or chart 7.1*
Col. 15 Vel. Full	$V_f = (0.590/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59/0.013)(1.5)^{0.67}\ (0.03)^{0.5}$ $V_f = 10.3\ ft/s$	*equation 7.3 or chart 7.1*
Col. 16 Vel. Design	$Q/Q_f = 5.1/18.1 = 0.28$ $V/V_f = 0.84$ $V = (0.84)\ (10.3)$ $V = 8.7\ ft/s$	 *chart 7.5*
Col. 17 Sect. Time	$T_s = L/V = 328\ /\ 8.75\ /\ 60$ $T_s = 0.6$ *min; use 1 min*	*Col. 3 divided by Col. 16*
Col. 20 Crown Drop	$= H_{ah} = K_{ah}\ V^2\ /\ (2g)$ $= (0.5)(8.7)^2\ /\ [(2)(32.2)]$ $= 0.6\ ft$	*equation 7.7 with table 7.3* $K_{ah} = 0.5$ *for inlet - straight run*
Col. 18 U/S Invert	$= 354.67 - 0.6$ $= 354.07\ ft$	*Downstream invert of upstream conduit minus estimated structure loss (drop)*
Col. 19 D/S Invert	$= (354.07) - (328)(0.03)$ $= 344.23\ ft$	*Col. 18 - (Col. 3)(Col. 21)*

Structure 42 to 43

Col. 1 From structure	= 42	
Col. 2 To structure	= 43	
Col. 3 Run Length	$L = 14.1$ ft	*figure 7A.2*
Col. 4 Inlet Area	$A_i = 0.32$ ac	*table 7A.1*
Col. 5 Total Area	$A_t = 0.32 + 0.99$ $A_t = 1.31$ ac	*Col. 4 plus structure 41 total area*
Col. 6 "C"	$C = 0.73$	*table 7A.1*
Col. 7 Inlet CA	$CA = (0.32)(0.73)$ $CA = 0.23$ ac	*Col. 4 times Col. 6*
Col. 8 Sum CA	$3CA = 0.23 + 0.72$ $3CA = 0.95$ ac	*Col. 7 plus structure 41 total CA values*
Col. 9 Inlet Time	$t_i = 2$ min	*table 7A.1*
Col. 10 Sys. Time	$t_c = 5$ min	*Col. 9 + Col. 17 for line 40-41 plus Col.17 for line 41-42*
Col. 11 Intensity	$I = 7.1$ in/hr	*From table 7A.2*
Col. 12 Runoff	$Q = (CA)\ (I)$ $Q = (0.95)\ (7.1)$ $Q = 6.75\ ft^3/sec)$	*Col. 8 times Col. 11*
Col. 21 Slope	$S = 0.001$	*Select desired pipe slope*
Col. 13 Pipe Dia.	$D = [(Qn)/(0.463\ S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.001)^{0.5}]^{0.375}$ $D = 1.96$ ft $D = 2.0$ ft	*equation 7.5 or chart 7.1* *Use nominal size*
Col. 14 Full Cap.	$Q_f = (0.463/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46/(0.013)(2.0)^{2.67}\ (0.001)^{0.5}$ $Q_f = 7.12\ ft^3/s$	*equation 7.4 or chart 7.1*
Col. 15 Vel. Full	$V_f = (0.590/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67}(0.001)^{0.5}$ $V_f = 2.28$ ft/s	*equation 7.3 or chart 7.1*
Col. 16 Vel. Design	$Q/Q_f = 6.75/7.12 = 0.95$ $V/V_f = 1.15$ $V = (1.15)\ (2.28)$ $V = 2.6$ ft/s	*chart 7.5*

Col. 17	*Sect. Time*	$t_s = L/V = 14.1 / 2.6 / 60$ $t_s = 0.09$ *min, use 0.0 min*	*Col. 3 divided by Col. 16*
Col. 20	*Crown Drop*	$= H_{ah} = K_{ah} V^2 / (2g)$ $= (1.5)(2.6)^2 / [(2)(32.2)]$ $= 0.16$ *ft*	*equation 7.7 and table 7.3;* $K_{ah} = 1.5$ *for inlet - angled through 90 degrees*
Col. 18	*U/S Invert*	*= 344.23 - 0.16* *= 344.07 ft conduit*	*Downstream invert of upstream minus estimated structure loss (drop)*
Col. 19	*D/S Invert*	*= 344.07 - (14.1)(0.001)* *= 344.06 ft*	*Col. 18 - (Col. 3)(Col. 21)*

<u>Structure 43 to 44</u>

Col. 1	*From*	*= 43*	
Col. 2	*To*	*= 44*	
Col. 3	*Run Length*	*L = 55.8 ft*	*figure 7A.2*
Col. 4	*Inlet Area*	$A_i = 0.0$ *ac*	*table 7A.1*
Col. 5	*Total Area*	$A_t = 1.31$ *ac*	*Col. 4 plus structure 42 total area*
Col. 6	*"C"*	*C = n/a*	*table 7A.1*
Col. 7	*Inlet CA*	*CA = 0.0*	*Col. 4 times Col. 6*
Col. 8	*Sum CA*	$\Sigma CA = 0.00 + 0.95$ $\Sigma CA = 0.95$ *ac*	*Col. 7 plus structure 42 total CA value*
Col. 9	*Inlet Time*	*n/a*	*No inlet*
Col. 10	*Sys. Time*	$t_c = 5$ *min*	*Col. 10 + Col. 17 for line 42-43*
Col. 11	*Intensity*	*I = 7.1 in/hr)*	*From table 7A.2*
Col. 12	*Runoff*	*Q = (CA) I* *Q = (0.95) (7.1)* $Q = 6.75$ ft^3*/sec*	*Col. 8 times Col. 11*
Col. 21	*Slope*	*S = 0.01*	*Select desired pipe slope*

Col. 13 Pipe Dia.	$D = [(Qn)/(0.463\ S_o^{0.5})]^{0.375}$ $D = [(6.75)(0.013)/(0.46)(0.01)^{0.5}]^{0.375}$ $D = 1.27$ *ft* D = 2.0 ft	*equation 7.5 or chart 7.1* *U/S conduit was 2.0 ft. – Do not reduce size inside the system*
Col. 14 Full Cap.	$Q_f = (0.463/n)(D^{2.67})(S_o^{0.5})$ $Q_f = (0.46)/(0.013)(2.0)^{2.67}\ (0.01)^{0.5}$ $Q_f = 22.52\ ft^3/s$	*equation 7.4 or chart 7.1*
Col. 15 Vel. Full	$V_f = (0.590/n)\ D^{0.67}\ S_o^{0.5}$ $V_f = (0.59)/(0.013)(2.0)^{0.67}(0.01)^{0.5}$ $V_f = 7.22$ *ft/s*	*equation 7.3 or chart 7.1*
Col. 16 Vel. Design	$Q/Q_f = 6.75/22.52 = 0.30$ $V/V_f = 0.84$ $V = (0.84)\ (7.22)$ $V = 6.1$ *ft/s*	 *chart 7.5*
Col. 17 Sect. Time	$t_s = 55.8\ /6.1\ /\ 60$ $t_s = 0.15$ *min, use 0.0 min*	*Col. 3 divided by Col. 16*
Col. 19 D/S Invert	*= 330.71 ft*	*Invert at discharge point in ditch*
Col. 18 U/S Invert	*= 330.71 + (55.8)(0.01)* *= 331.27 ft*	*Col. 19 + (Col. 3)(Col. 21)*
Col. 20 Crown Drop	*= 344.06 - 331.27* *= 12.79 ft*	*Col. 19 previous run – Col. 18* *straight run*

(2) Energy Grade Line Evaluation Computations

The following computational procedure follows the steps outlined in Section 7.10 above. Starting at structure 44, computations proceed in the upstream direction. A summary tabulation of the computational process is provided in Figure 7B.4 and Figure 7B.5. The column by column computations for each section of storm drain follow:

RUN FROM STRUCTURE 44 TO 43

Outlet

Step 1	*Col. 1A*	*Outlet*	
	Col. 14A	*HGL = 333.0*	*Downstream pool elevation*
	Col. 10A	*EGL = 333.0*	*Assume no velocity in pool*

Structure 44

Step 2	*Col. 1A, 1B*	*Str. ID = 44*	*Outlet*
	Col. 15A	*Invert = 330.71 ft*	*Outfall invert*
		TOC = 330.71 + 2.0	*Top of storm drain at outfall*
		TOC = 332.71	
		Surface Elev = 332.71	*Match TOC*
Step 3		*HGL = TW = 333.0*	*From Step 1*
		$EGL_i = HGL + V^2/2g$	*Use Case 1 since TW is above*
		the top of conduit	
		$EGL_i = 333.0 + 0.07$	
	Col. 13A	$EGL_i = 333.07$	EGL_i *for str. 44*

Structure 43

Step 4	*Col. 1A, 1B*	*Str. ID = 43*	*Next Structure*
	Col. 2A	*D = 2.0 ft*	*Pipe Diameter*
	Col. 3A	*Q = 6.75 cfs*	*Conduit discharge (design value)*
	Col. 4A	*L = 55.8 ft*	*Conduit length*
Step 5	*Col. 5A*	$V = Q/A$	*Velocity; use full barrel velocity*
		$V = 6.75/[(\pi/4)\,(2.0)^2]$	*since outlet is submerged.*
		$V = 2.15$ *ft/s*	
	Col. 7A	$V^2/2g = (2.15)^2/(2)(32.2)$	*Velocity head in conduit*
		$= 0.07$ *ft*	
Step 6	*Col. 8A*	$S_f = [(Qn)/(0.463D^{2.67})]^2$	*equation 7.12.*
		$S_f = [(6.75)(0.013)/(0.463)(2.0)^{2.67}]^2$	
		$S_f = 0.00090$ *ft/ft*	
Step 7	*Col. 2B*	$H_f = S_f L$	*equation 7.10*
		$H_f = (0.0009)\,(55.8)$	*Col. 8A x Col. 4A*
		$H_f = 0.05$	
	Col. 7B &	$h_b, H_c, H_e, H_j = 0$	
	Col. 9A	*Total = 0.05 ft*	

ENERGY GRADE LINE COMPUTATION SHEET - TABLE A
(English Solution)

COMPUTED BY ________ DATE ________ ROUTE ____________
CHECKED BY ________ DATE ________ SECTION ____________
PAGE ________ OF ________ COUNTY ____________
INITIAL TAILWATER ELEV. ____________

Str. ID (1)	D (ft) (2)	Q (ft³/s) (3)	L (ft) (4)	V (fps) (5)	d (ft) (6a)	d_c (6b)	$V^2/2g$ (ft) (7)	S_f (ft / ft) (8)	Total Pipe Loss (table B) (ft) (9)	EGL_o (ft) (10)	K table B (11)	$K(V^2/2g)$ (ft) (12)	EGL_i (ft) (13)	HGL (ft) (14)	U/S TOC (ft) (15)	Surf. Elev. (ft) (16
OUTLET										333.00				333.00		
44													333.07		332.71	332.71
43	2.0	6.75	55.8	2.15	FULL	n/a	0.07	0.0009	0.05	333.12	0.5	0.04	333.16	*333.16	346.06	347.76
											*(Drop Structure)					
43	(New Outlet)			2.6		0.8	0.10						345.56	345.46	346.06	347.76
		(Above calculations are for inlet end of STR 43 for conduit 42-43)														
42	2.0	6.75	14.1	2.6	1.56	0.80	0.10	0.001	0.014	345.57	0.62	0.06	346.11	346.01	345.73	349.31
41	1.5	5.10	328.0	8.65	0.56	0.85	1.16	-	0	355.79	-	-	355.98	355.10	356.17	360.0
40	1.5	3.35	361.0	7.52	0.43	0.70	0.88		0	-	0	0	366.50	366.50	367.0	370.0

Figure 7A.4. Energy grade line computation sheet, table A.

ENERGY GRADE LINE COMPUTATION SHEET - TABLE B
(English Solution)

COMPUTED BY ______ DATE ______ ROUTE ______
CHECKED BY ______ DATE ______ SECTION ______
PAGE ______ OF ______ COUNTY ______

	Pipe Losses (ft)						Structure Losses (ft)							
Str. ID (1)	H_f (2)	h_b (3)	H_c (4)	H_e (5)	H_j (6)	Total (7)	d_{aho} (8)	K_o (9)	C_D (10)	C_d (11)	C_Q (12)	C_p (13)	C_B (14)	K (15)
44														
	0.05					0.05								
43							1.89							0.5
	0.014					0.014								
42							1.40	1.55	1.0	0.40	1.0	1.0	1.0	0.62
						0.0								
41														
														0.0
40							1.0							

Figure 7A.5. Energy grade line computation sheet, table B.

Step 8 Col. 10A	$EGL_o = EGL_i$ + pipe loss $EGL_o = 333.07 + 0.05$ $EGL_o = 333.12$ ft	
	HGL =333.12 - 0.07 = 333.05	 *Check for full flow - close*
	TOC = 331.27 + 2.0 = 333.27	 *Assumption OK*
Step 9 Col. 8B	*Not applicable due to drop structure*	
Step 10 Col. 9B *and 11A*	$K_e = 0.5$	*Inflow pipe invert much higher than* d_{aho}. *Assume square edge entrance*
Step 17 Col. 12A	$K(V^2/2g) = (0.50)(0.07)$ $K(V^2/2g) = 0.04$ ft	*Col. 11A times Col. 7A*
Step 18 Col. 13A	$EGL_i = EGL_o$ $EGL_i = 333.12 + 0.04$ $EGL_i = 333.16$ ft	*Col 10A plus 12A*
Step 19 Col. 14A	$HGL = EGL_i = 333.16$ ft	*For drop structures, the HGL is the same as the EGL*
	d_{aho} = HGL- invert = 333.16 - 331.27 = 1.89 ft	 *Col. 8B*
Step 20 Col. 15A	*U/S TOC = Inv. + Dia.* *U/S TOC = 344.06 + 2.0* *U/S TOC = 346.06 ft*	*From storm drain comp. sheet (figure 7A.3)*
Step 21 Col. 16A	*Surf. Elev. = 347.76 ft* *347.76 > 333.09*	*From figure 7A.2.* *Surface elev. exceeds HGL, OK*
Step 2 Col. 1A, 1B *Col. 15A* *Col. 16A*	*Str. ID = 43* *U/S TOC = 344.06 + 2.0* *= 346.06* *Surface Elev = 347.76*	*Drop Structure - new start*
Step 3 *Col. 14A*	$HGL' = inv. + (d_c+D)/2$ $HGL' = 344.06 +$ $(0.80 + 2.0)/2$ $HGL = 345.46$ ft	*Calculate new HGL - Use Case 2* d_c *from Chart 7.2*
  Col. 13A	$EGL = HGL + V^2/2g$ $EGL = 345.46 + 0.10$ $EGL = 345.56$ ft	*V = 2.6 fps from Prelim. Comp. Sht.*

Structure 42

Step 4	*Col.1A*	*Str. ID = 42*	
	Col. 2A	*D = 2.0 ft*	*Pipe Diameter*
	Col. 3A	*Q = 6.75 cfs*	*Conduit discharge (design value)*
	Col. 4A	*L = 14.1 f*	*Conduit length*
Step 5A	*Col. 5A*	*V = 2.6 ft/s*	*For Flow: Actual velocity from*
		Q/Qf = 6.75 / 7.12 = 0.95	*storm drain computation sheet.*
	Col. 6A	$d_n = 1.56$ *ft*	*Chart 7.5*
	Col. 7A	$V^2/2g = (2.6)^2/(2)(32.2)$	*Velocity head in conduit*
		$V^2/2g = 0.10$ *ft*	
Step 5B	*Col.6bA*	$d_c = 0.80$ *ft*	*From Chart 7.2*
Step 5C		$d_n < d_c$	*Flow is subcritical*
Step 6	*Col. 8A*	$S_f = 0.001$	*Conduit not full so* S_f *= pipe slope*
			$d_n = 1.56$ *(Chart 7.5)*
			$d_c = 0.80$ *(Chart 7.2)*
			Flow is subcritical
Step 7		$H_f = S_f L$	*equation 7.10*
		$H_f = (0.001)\ (14.1)$	*Col. 8A x Col. 5A*
	Col. 2B	$H_f = 0.014$ *ft*	
		$h_b, H_c, H_e, H_j = 0$	
	Col. 7B and 9A	*Total = 0.014 ft*	
Step 8		$EGL_o = EGL_i +$ *total pipe loss*	
			Col. 14A plus Col. 9A
		$EGL_o = 345.56 + 0.014$	
	Col. 10A	$EGL_o = 345.57$ *ft*	
Step 9	*Col. 8B*	$d_{aho} = EGL_o$ *- velocity head - pipe invert*	
			Col. 10A - Column 7A - pipe invert
		$d_{aho} = 345.57 - 0.10 - 344.07$	
		$d_{aho} = 1.40$ *ft*	
Step 10	*Col. 9B*	$K_o = 0.1(b/D_o)(1-\sin\theta) + 1.4(b/D_o)^{0.15}\sin(\theta)$	*equation 7.18*
		$b = 4.0$ *ft*	*Access hole diameter.*
		$D_o = 2.0$ *ft*	*Col. 2A - outlet pipe diam*
		$\theta = 90E$	*Flow deflection angle*
		$K_o = 0.1\ (4.0/2.0)(1 - \sin 90) + 1.4(4.0/2.0)^{0.15}\sin 90$	
		$K_o = 1.55$	

Step 11 Col. 10B	$C_D = (D_o/D_i)^3$ $d_{aho} = 1.40$ $d_{aho}/D_o = (1.40/2.0)$ $d_{aho}/D_o = 0.70 < 3.2$ $C_D = 1.0$	*equation 7.19; pipe diameter* *Column 8B* *therefore*
Step 12 Col. 11B	$C_d = 0.5\ (d_{aho}/\ D_o)^{0.6}$ $d_{aho}/D_o = 0.70 < 3.2$ $C_d = 0.5\ (1.4/2.0)^{0.6}$ $C_d = 0.40$	*equation 7.20; Flow depth correction.*
Step 13 Col. 12B	$C_Q = (1\text{-}2 \sin \theta)(1\text{-}Q_i/Q_o)^{0.75}+1$ $C_Q = 1.0$	*equation 7.21; relative Flow* *No additional pipes entering*
Step 14 Col. 13B	$C_p = 1+0.2(h/D_o)[(h\text{-}d)/D_o]$ $C_p = 1.0$	*equation 7.22; plunging Flow* *No plunging Flow*
Step 15 Col. 14B	$C_B = 1.0$	*Benching Correction, flat floor (table 7.6)*
Step 16 Col. 15B and 11A	$K = K_oC_DC_dC_QC_pC_B$ $K = (1.55)(1.0)(0.40)(1.0)(1.0)(1.0)$ $K = 0.62$	*equation 7.17*
Step 17 Col. 12A	$K(V^2/2g) = (0.62)(0.10)$ $K(V^2/2g) = 0.06$ ft	*Col. 11A times Col. 7A*
Step 18 Col. 13A	$EGL_i = EGL_o + K(V^2/2g)$ $EGL_i = 346.05 + 0.06$ $EGL_i = 346.11$	*Col. 10A plus 12A*
Step 19 Col. 14A	$HGL = EGL_i - V^2/2g$ $HGL = 346.11 - 0.10$ $HGL = 346.01$ ft	*Col. 13A minus Col. 7A*
Step 20 Col 15A	*U/S TOC = Inv. + Dia.* *U/S TOC = 344.23 + 1.5* *U/S TOC = 345.73 ft*	*Information from storm drain comp.sheet (figure 7A.3)*
Step 21 Col 16A	*Surf. Elev. = 349.31 ft* $349.31 > 345.96$	*From figure 7A.2* *Surface elev. exceeds HGL, OK*

<u>Structure 41</u>

Step 4 Col. 1A, 1B	*Str. ID = 41*	*Next Structure*
Col. 2A	*D = 1.50 ft*	*Pipe Diameter*
Col. 3A	*Q = 5.10 cfs*	*Conduit discharge (design value)*
Col. 4A	*L = 328 ft*	*Conduit length*

Step 5 Part full Flow from column's 12 and 15 of storm drain computation sheet.		*Continue with Step 5A*
Step 5A	$Q/Q_f = 5.1/18.1 = 0.28$	
	$d/d_f = 0.37$	*Chart 7.5*
	$d = (0.37)\,(1.5)$	
Col. 6a	$d = 0.56$ *ft*	
	$V/V_f = 0.84$	
Chart 7.5		
	$V = (0.84)(10.3)$	
Col. 5A	$V = 8.65$ *fps*	
	$V^2/2g = (8.65)^2/(2)(32.2)$	*Velocity head*
Col. 7A	$V^2/2g = 1.16$ *ft*	
Step 5B Col. 6bA	$d_c = 0.85$ *ft*	*Chart 7.2*
Step 5C	$0.56 < 0.85$	*Supercritical Flow since* $d_n < d_c$
Step 5DCol. 7B	*Total pipe loss = 0*	

Structure 40

Step 5E		
Col. 1A,1B	*Str. Id. = 40*	*Next structure*
Col. 2A	$D = 1.5$ *ft*	*Pipe diameter*
Col. 3A	$Q = 3.35$ *cfs*	*Conduit discharge (design)*
Col. 4A	$L = 361.0$ *ft*	*Conduit length*
Step 5F	$Q/Q_f = 3.3/18.1 = 0.18$	
	$d/d_c = 0.29$	*Chart 7.5.*
	$d = (0.29)(1.5)$	
Col. 6aA	$d = 0.43$ *ft*	
Col. 6bA	$d_c = 0.7$ *ft*	*Chart 7.2*
Step 5H	$V/V_f = 0.73$	*Chart 7.5*
	$V = (0.73)(10.3)$	
Col. 5A	$V = 7.52$ *fps*	
	$V^2/2g = (7.52)^2/(2)(32.2)$	*Velocity head*
Col. 7A	$V^2/2g = 0.88$ *ft*	
Step 5I	$d_n = 0.43 \text{ ft} < 0.70 \text{ ft} = d_c$	*Supercritical Flow since* $d_n < d_c$

Step 5K Col. 11A, and 15B	*$K = 0.0$*	*Str. 41 line; supercritical Flow;*
Col. 12A	*$K(V^2/2g) = 0$*	*no structure losses*

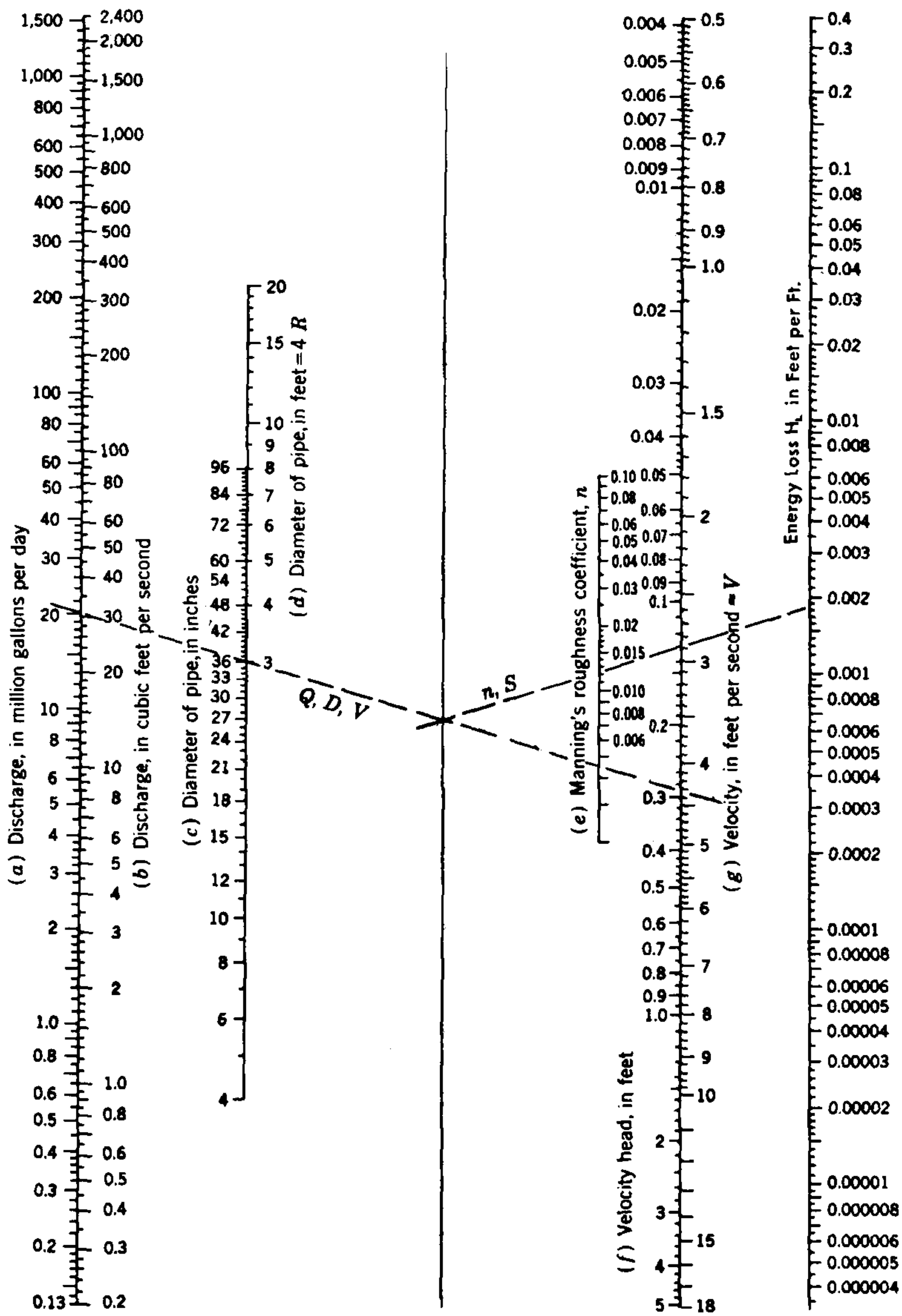
Since both conduits 42-41 and 41-40 are supercritical - establish HGL and EGL at each side of access hole 41.

	HGL = Inv. + d	
	HGL = 354.07 + 0.56	*D/S Invert + Flow depth*
	HGL = 354.63 ft	
	EGL = 354.63 + 1.16	*HGL + velocity head*
Col. 10A	*EGL = 355.79 ft*	*EGL_o of Str.41*
	HGL = 354.67 + 0.43	*U/S invert + Flow depth*
Col. 14A	*HGL = 355.10 f*	*Highest HGL*
	EGL = 355.10 + 0.88	*HGL + velocity head*
Col. 13A	*EGL = 355.98 ft*	*EGL_i of Str. 41*
Step 20 Col. 15A	*U/S TOC = Inv. + Dia.*	*Information from storm drain comp.*
	U/S TOC = 354.67 + 1.5	*Sheet (fig 7A.3) for Str. 41*
	U/S TOC = 356.17 ft	
Step 21 Col. 16A	*Surf. Elev. = 360.0 ft*	*From figure 7A.2.*
	360.0 > 355.10	*Surface elev. > HGL, OK*
Step 10b Col. 8B	*d_{aho} = 0.67 (1.5) = 1.0 ft*	*Chart 7.3. HW/D = 0.67*
	HGL = Str. 40 Inv. + d_{aho}	*Structure Inv. from storm*
	HGL = 365.50 + 1.0	*drain comp. sheet.*
Col. 14A	*HGL = 366.50 ft*	
Col.13A	*EGL = 366.50 ft*	*Assume no velocity in str.*
Step 20 Col. 15A	*U/S TOC = Inv. + Dia.*	*Information from storm*
	U/S TOC = 365.5 + 1.5	*drain comp. sheet (figure*
	U/S TOC = 367.0 ft	*7A.3) for Str. 40.*
Step 21 Col. 16A	*Surf. Elev. = 370.0 ft*	*From figure 7A.2.*
	370.0 ft > 366.50 ft	*Surface Elev. > HGL, OK*

See figures 7A.4 and 7A.5 for the tabulation of results. The final HGL values are indicated in figure 7A.2.

7.15 APPENDIX B – Design Charts

CHART 7.1

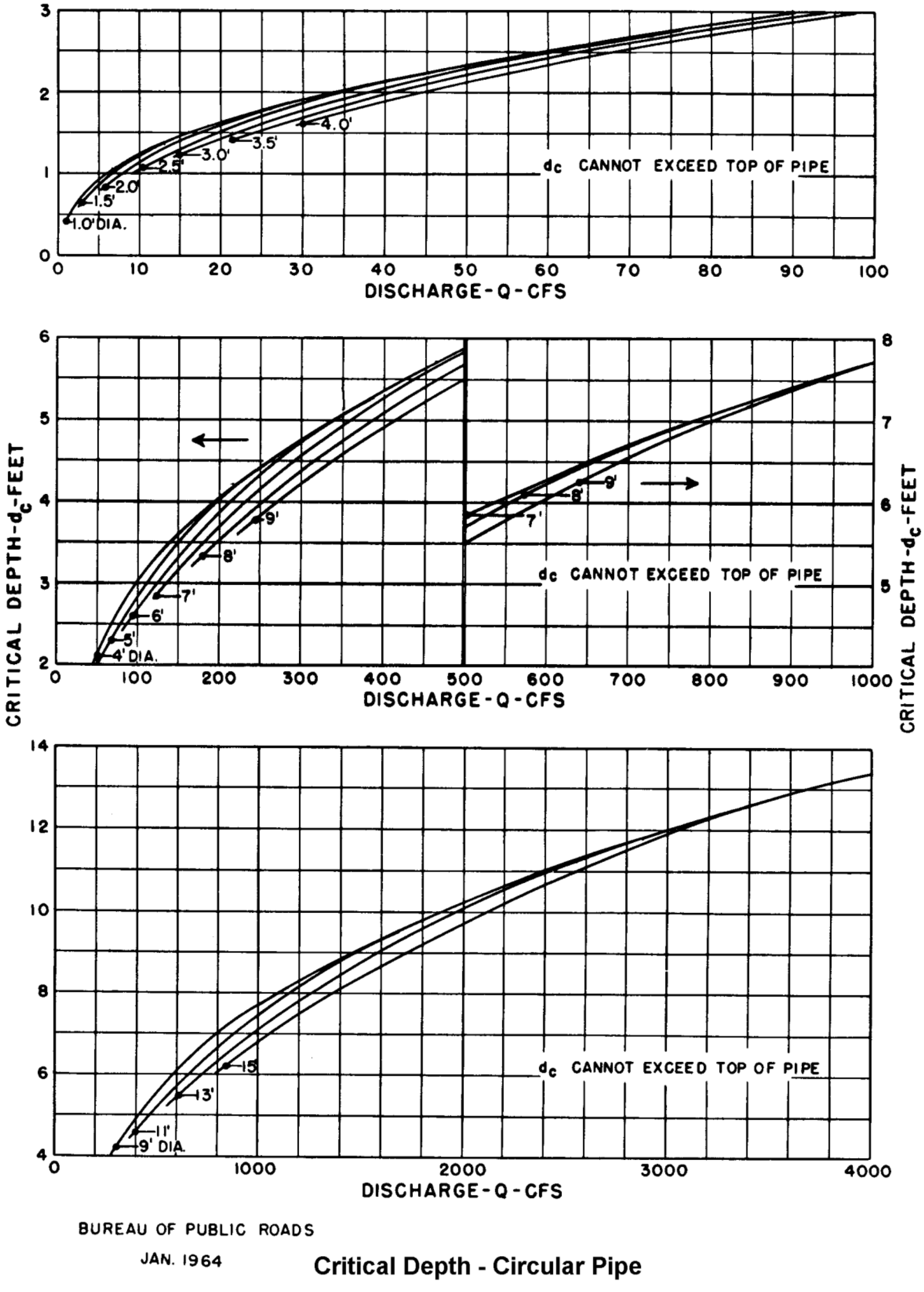


Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Solution of Manning's Equationfor Flow in Strom Drains

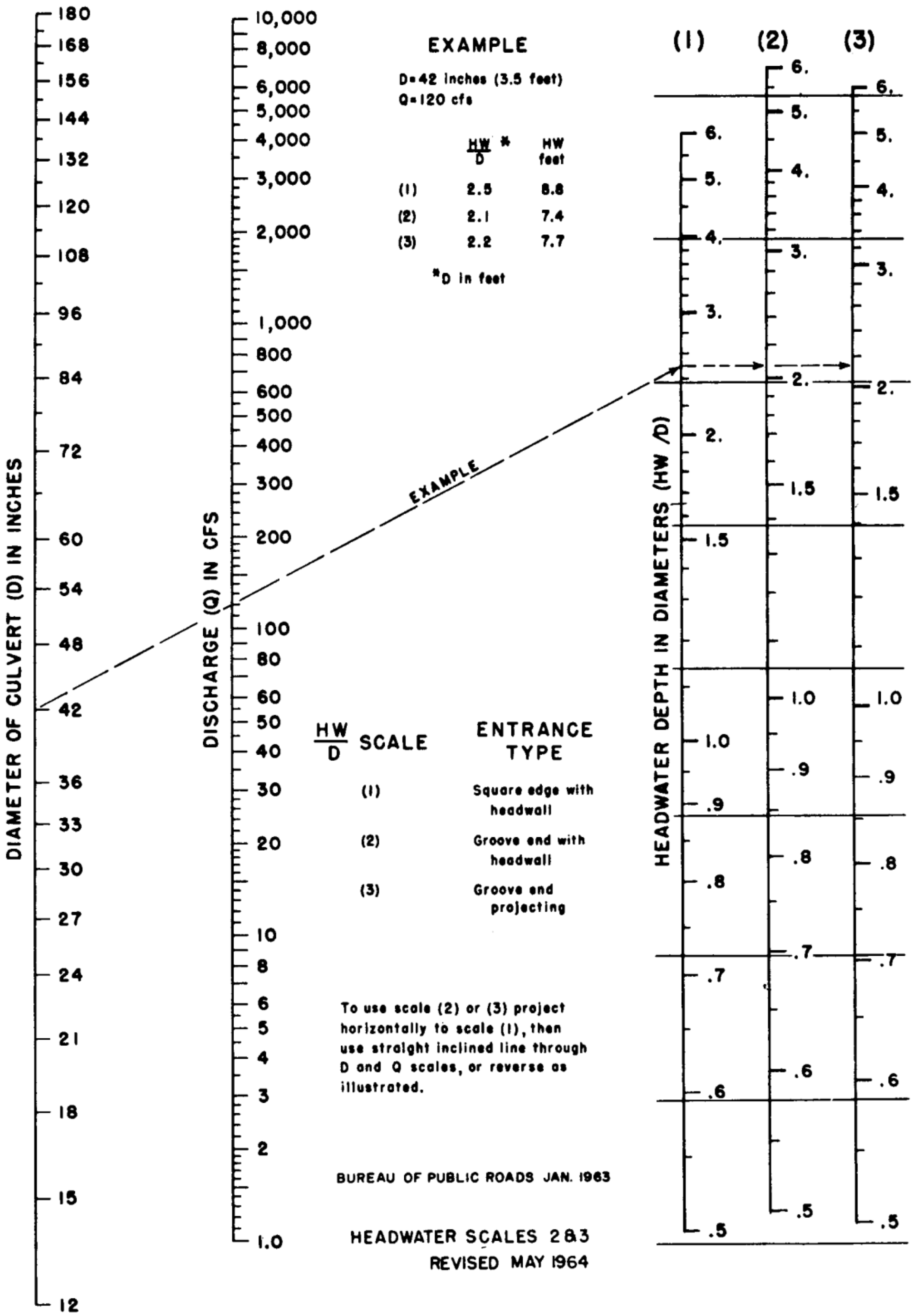
(Taken form "Modern Sewer Design" by American Iron and Steel Institute)

CHART 7.2



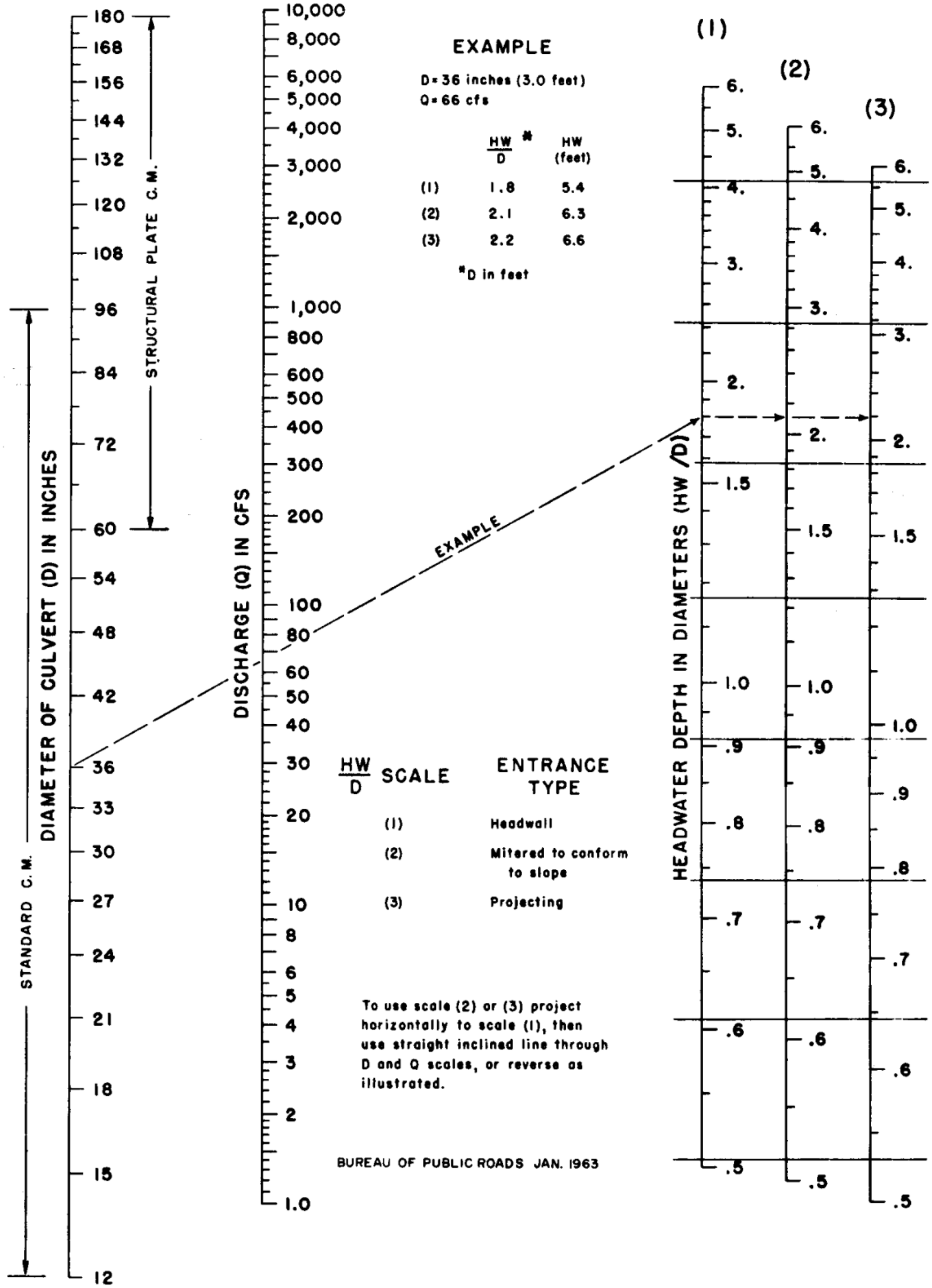
Critical Depth - Circular Pipe

CHART 7.3



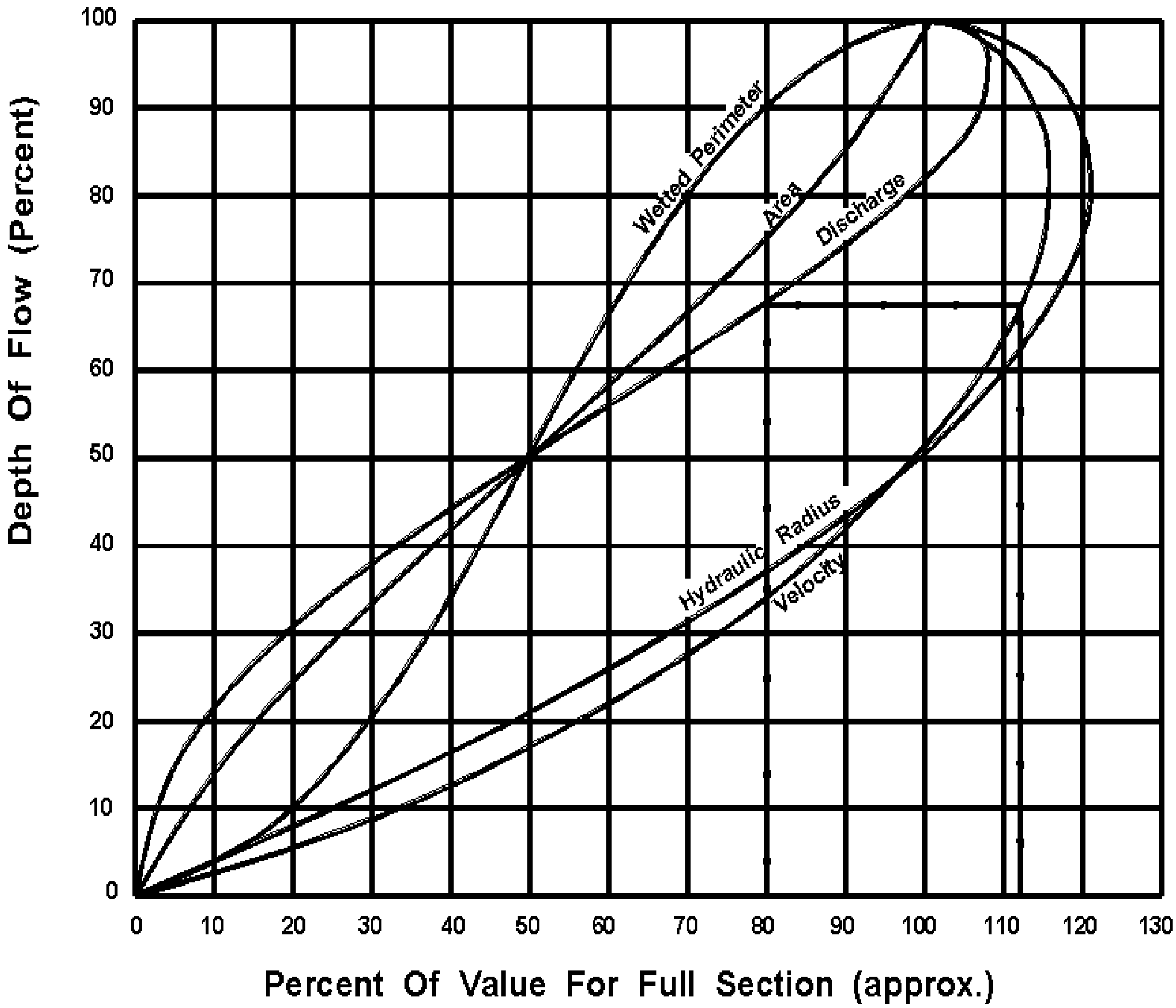
Headwater Water Depth for Concrete Pipe Culverts with Inlet Control - English Units

CHART 7.4



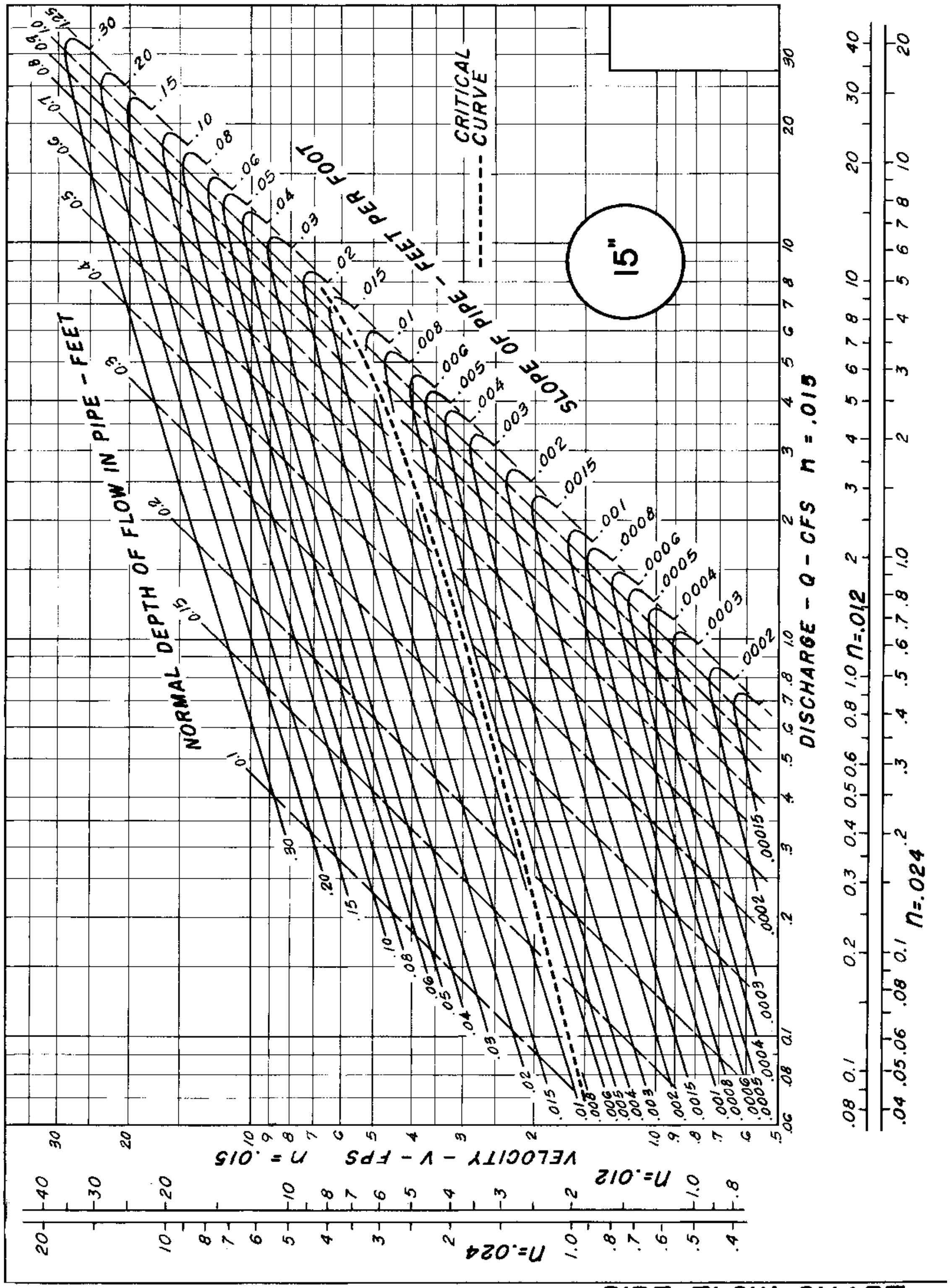
Headwater Depth for C. M. Pipe Culverts with Inlet Control - English Units

CHART 7.5



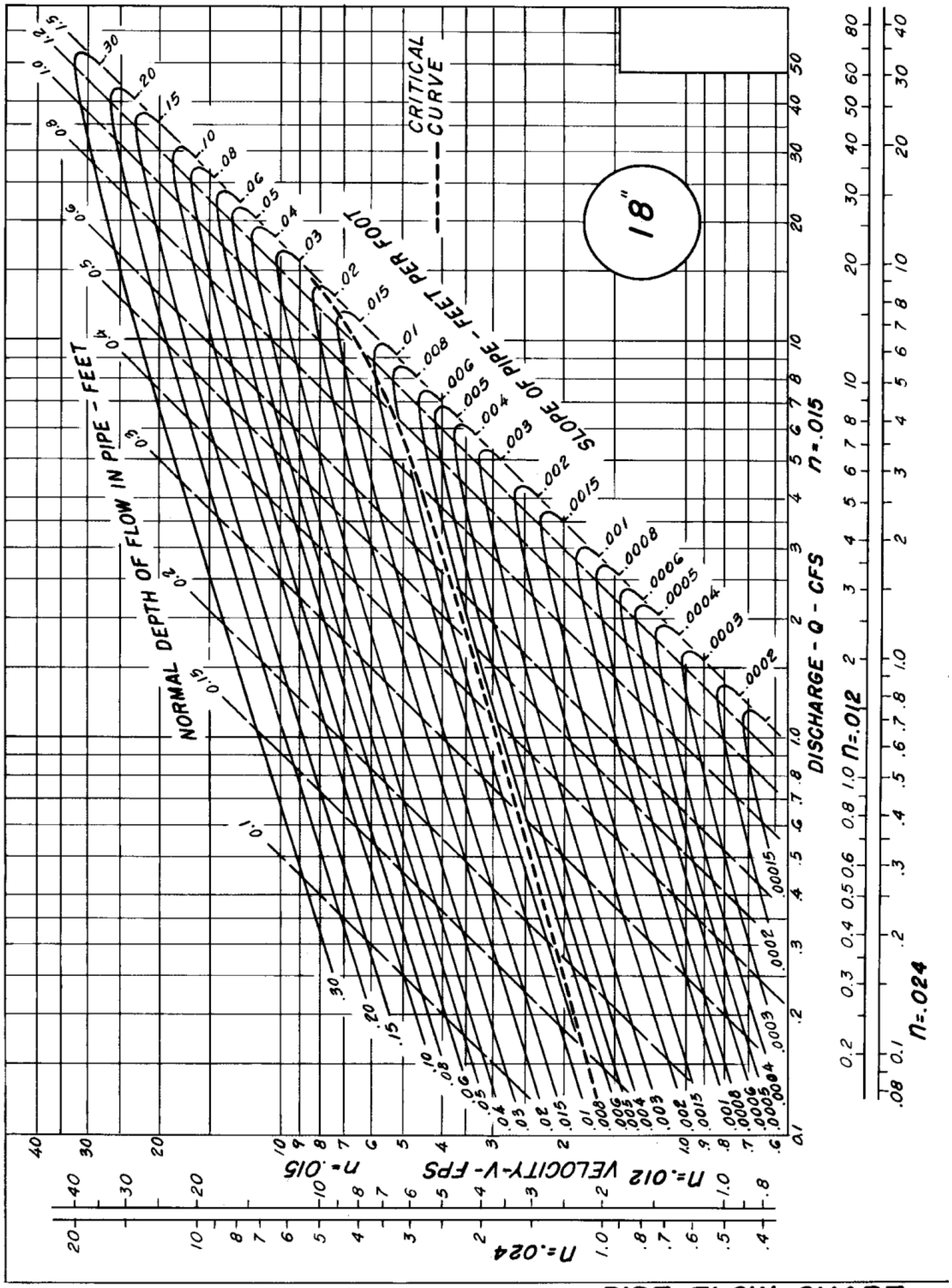
Hydraulic Elements Chart

DESIGN CHARTS FOR OPEN CHANNEL FLOW (FROM HDS-3[6])



PIPE FLOW CHART
15-INCH DIAMETER

Chart 7.6



PIPE FLOW CHART
18-INCH DIAMETER

Chart 7.7

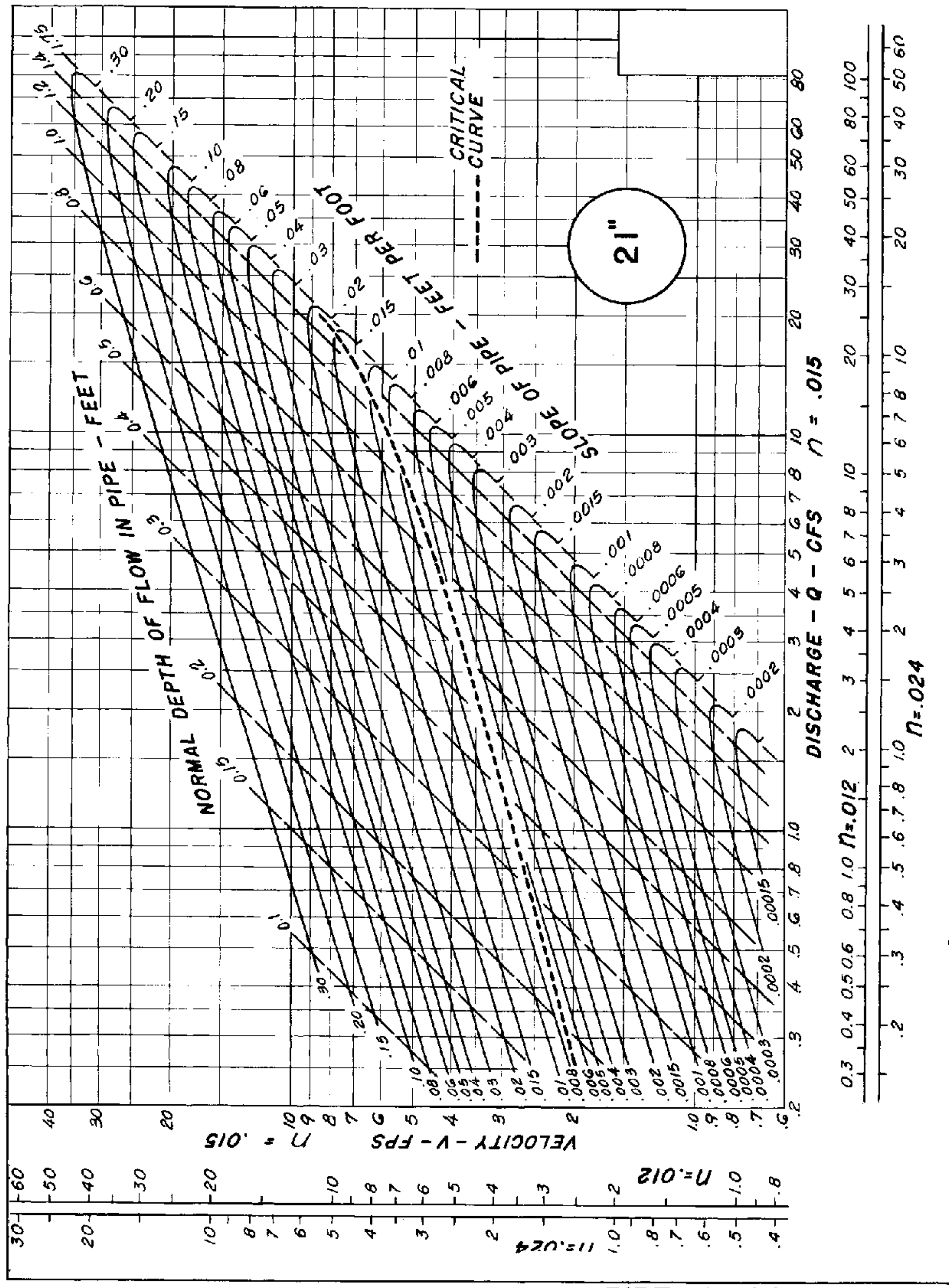


Chart 7.8

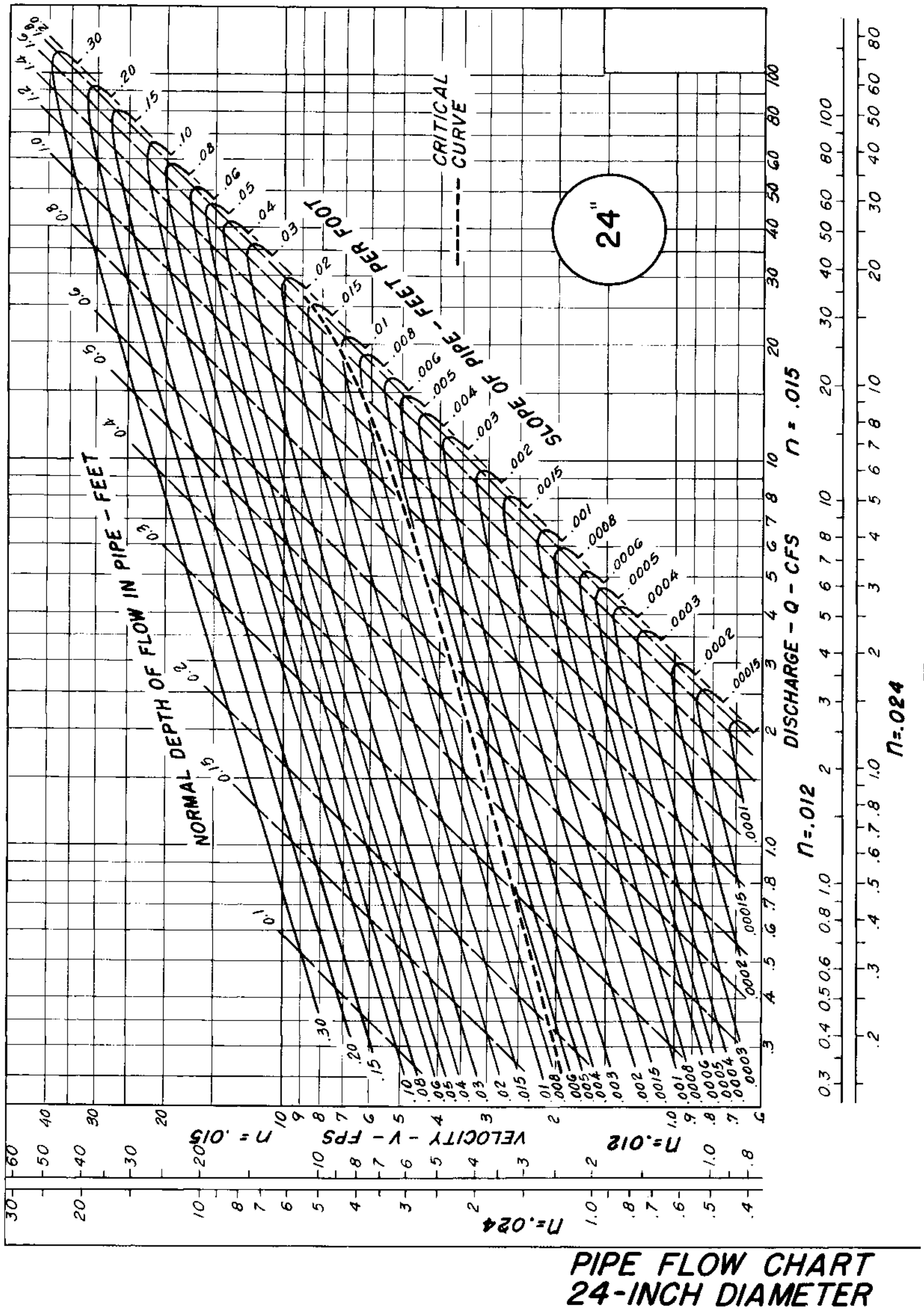
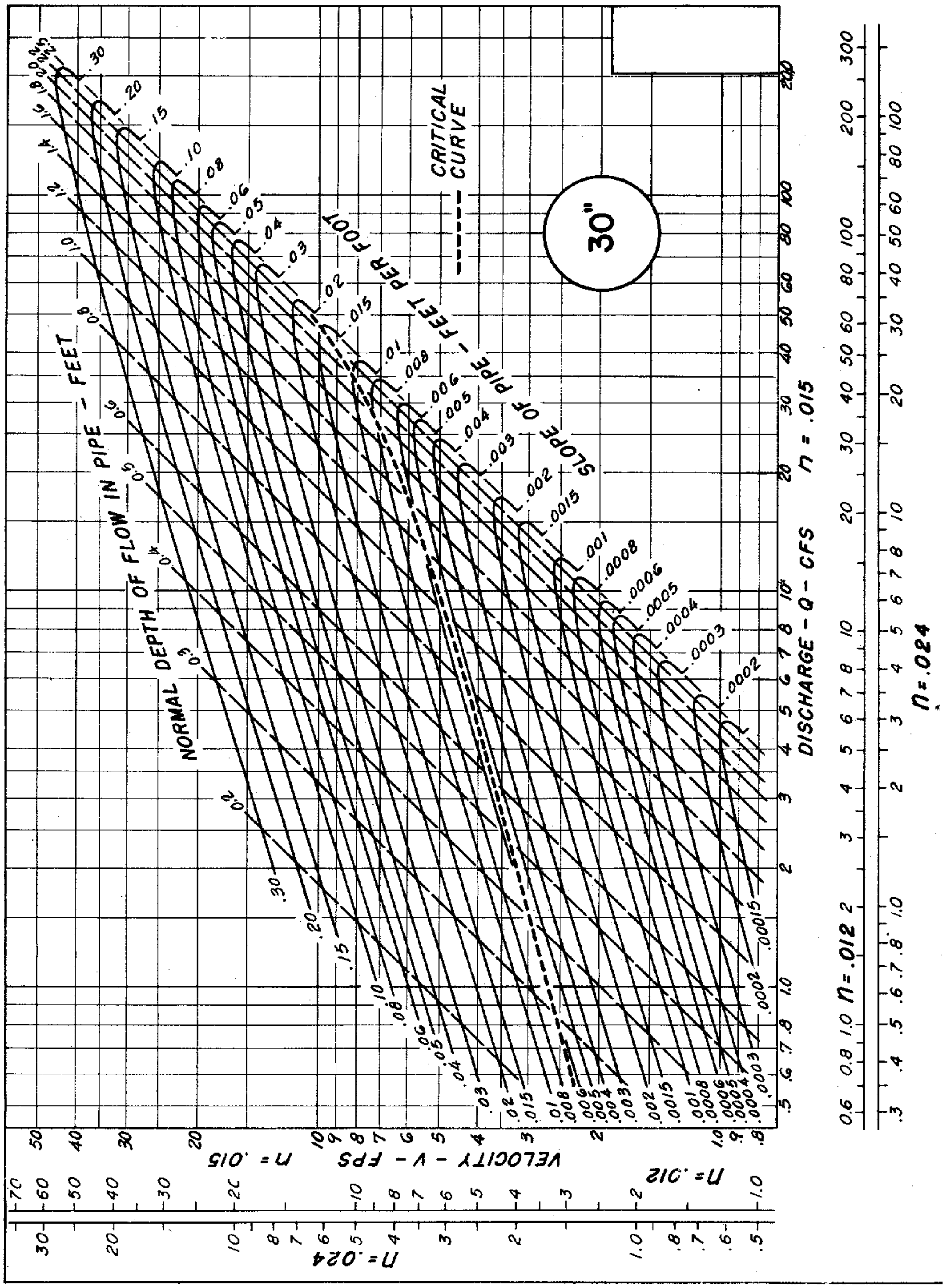
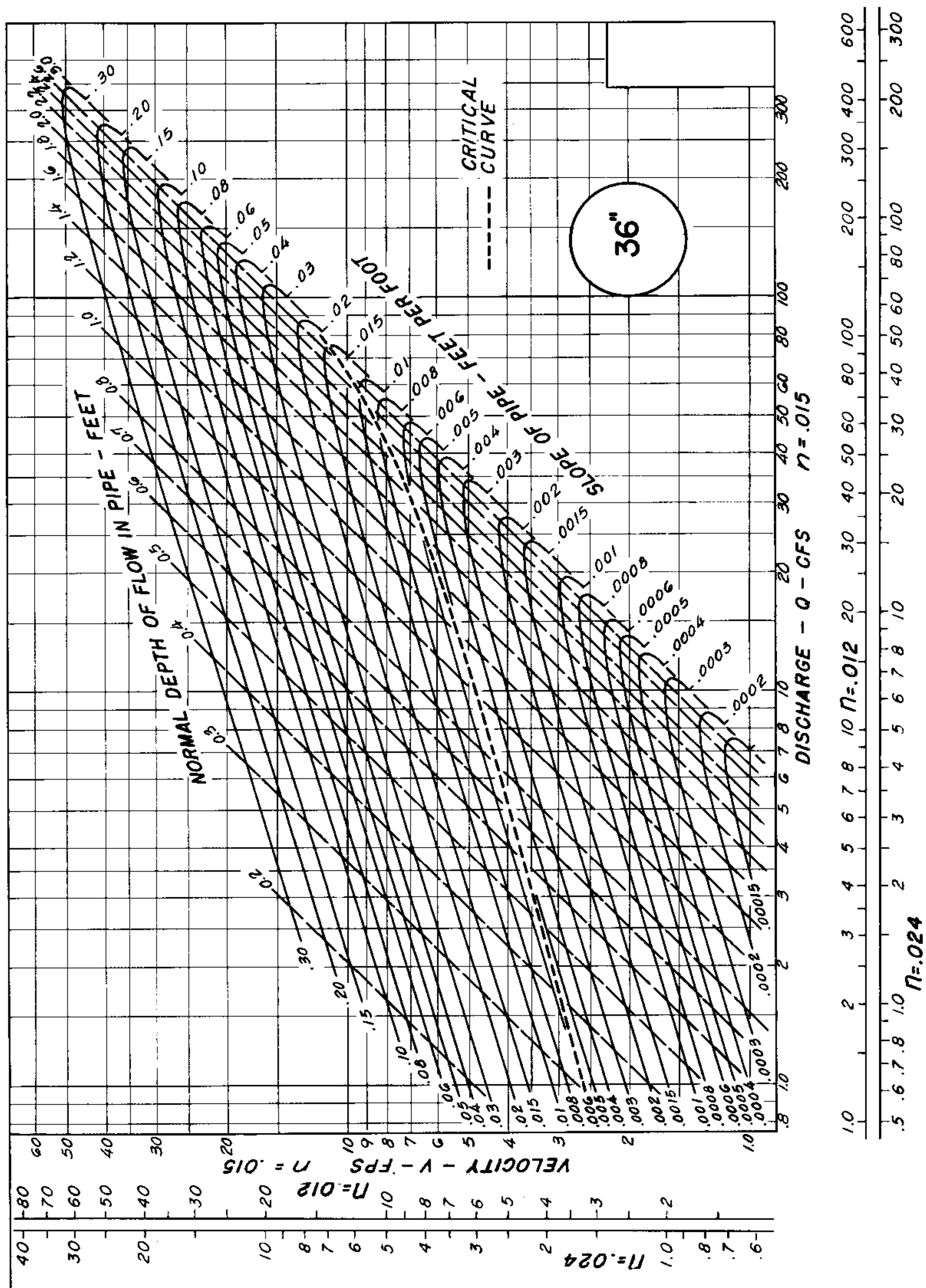

PIPE FLOW CHART
24-INCH DIAMETER
24"
CRITICAL CURVE
SLOPE OF PIPE - FEET PER FOOT
NORMAL DEPTH OF FLOW IN PIPE - FEET
DISCHARGE - Q - CFS
VELOCITY - V - FPS
n = .015
n=.012
n=.024

Chart 7.9



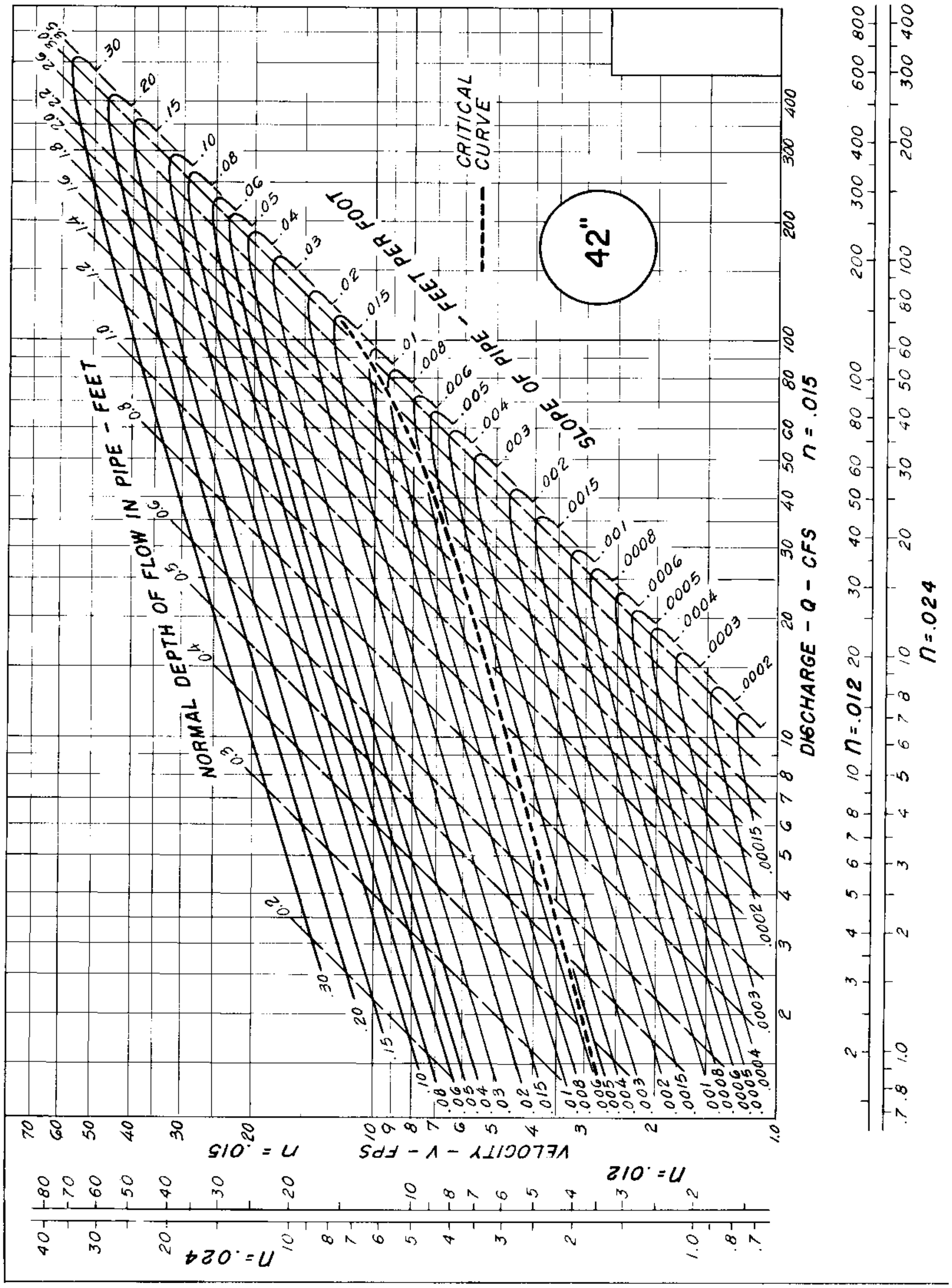
PIPE FLOW CHART
30-INCH DIAMETER

Chart 7.10



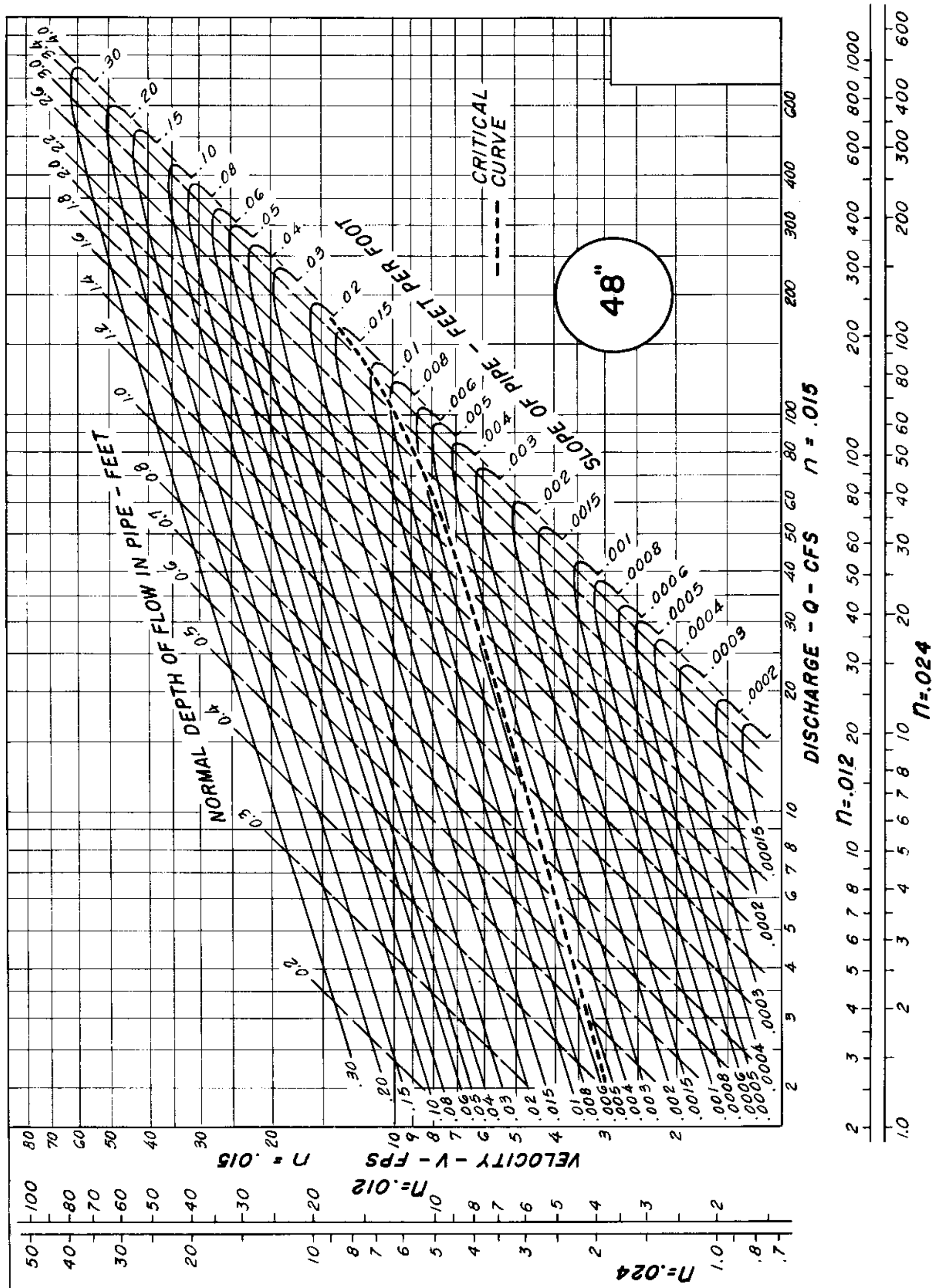
PIPE FLOW CHART
36-INCH DIAMETER

Chart 7.11



PIPE FLOW CHART
42-INCH DIAMETER

Chart 7.12



PIPE FLOW CHART
48-INCH DIAMETER

Chart 7.13

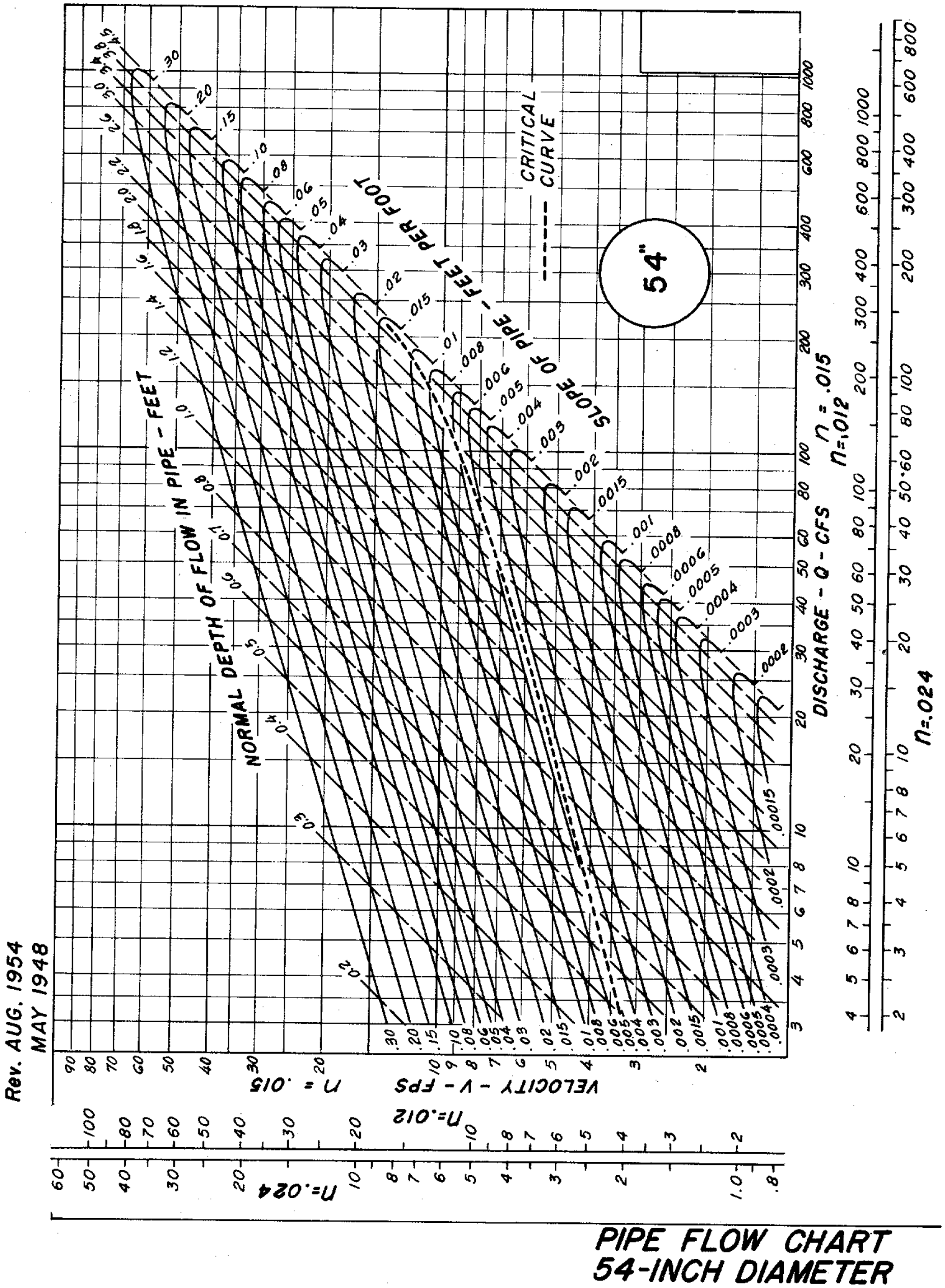
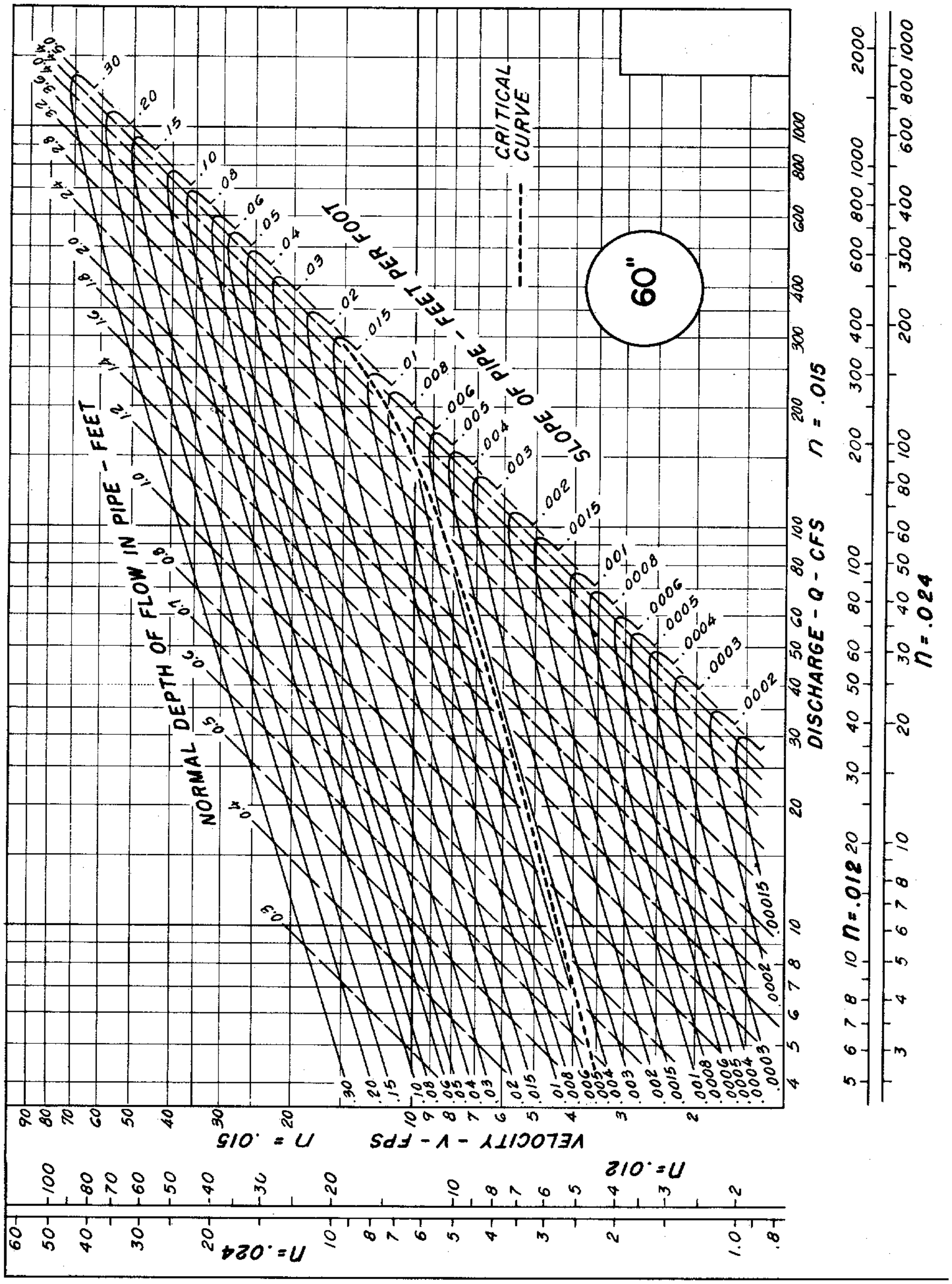
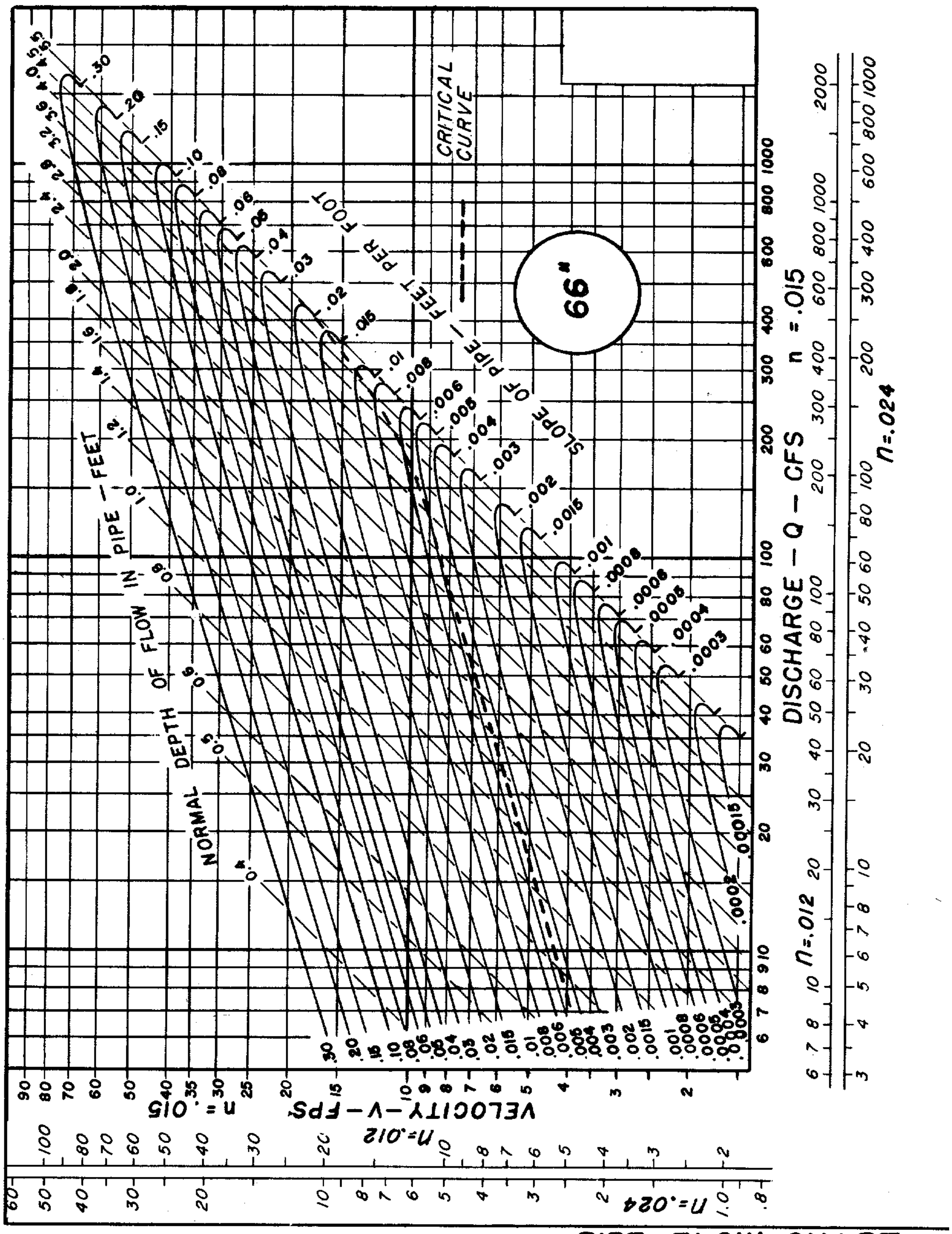

Rev. AUG. 1954
MAY 1948
NORMAL DEPTH OF FLOW IN PIPE - FEET
SLOPE OF PIPE - FEET PER FOOT
CRITICAL CURVE
54"
DISCHARGE - Q - CFS
VELOCITY - V - FPS
n = .015
n=.012
n=.024
PIPE FLOW CHART
54-INCH DIAMETER

Chart 7.14



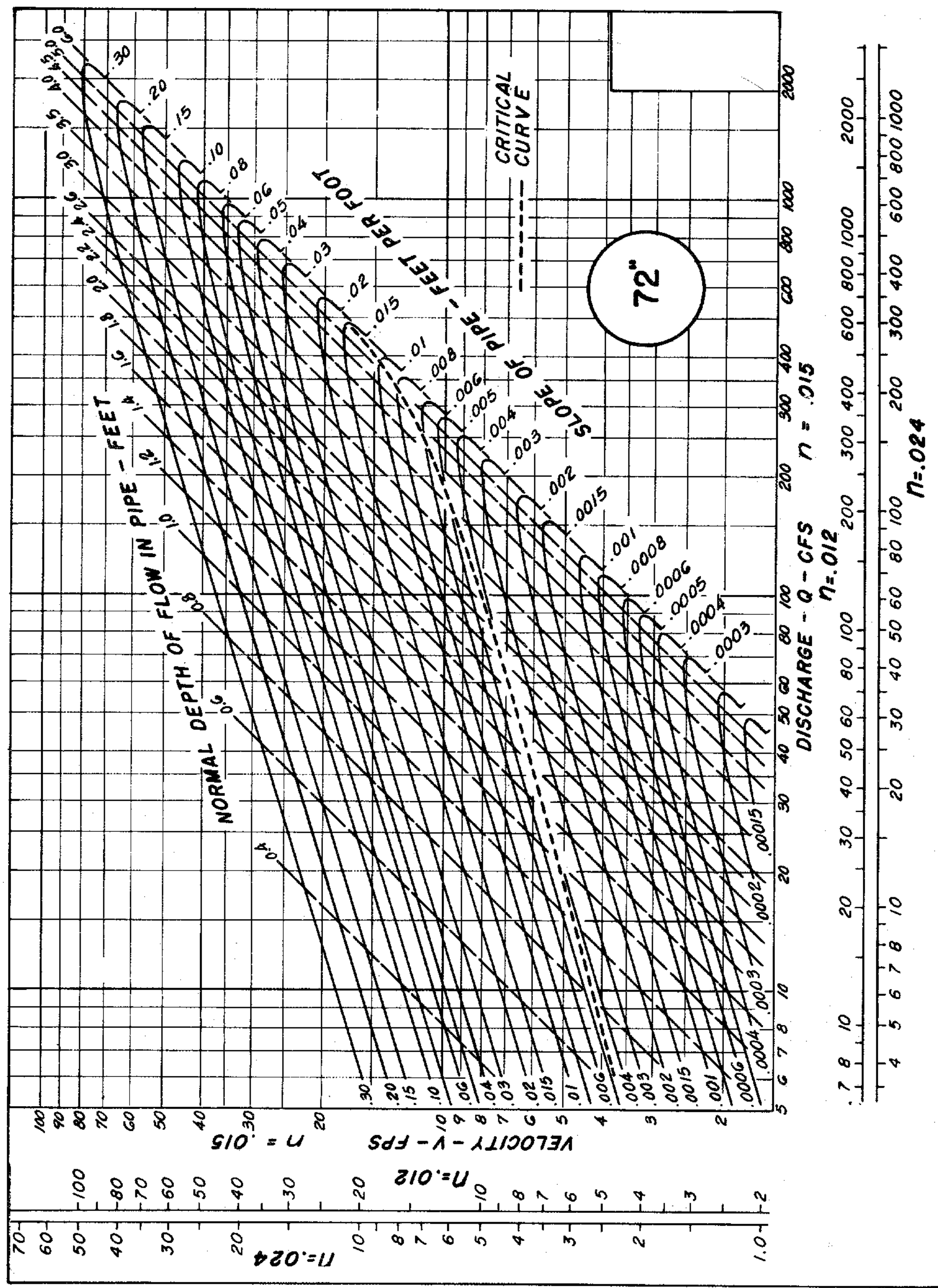
PIPE FLOW CHART
60-INCH DIAMETER

Chart 7.15



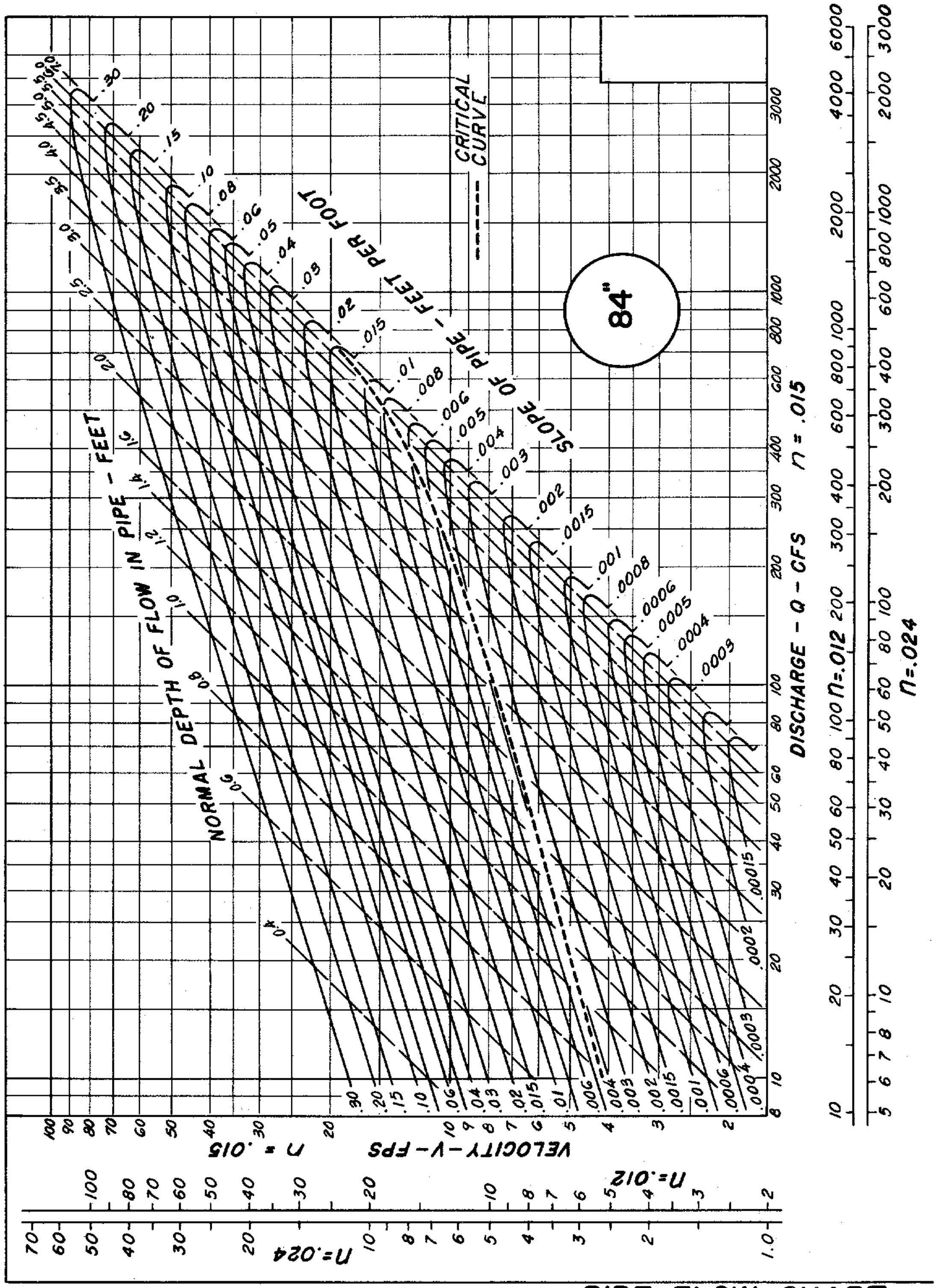
PIPE FLOW CHART
66-INCH DIAMETER

Chart 7.16



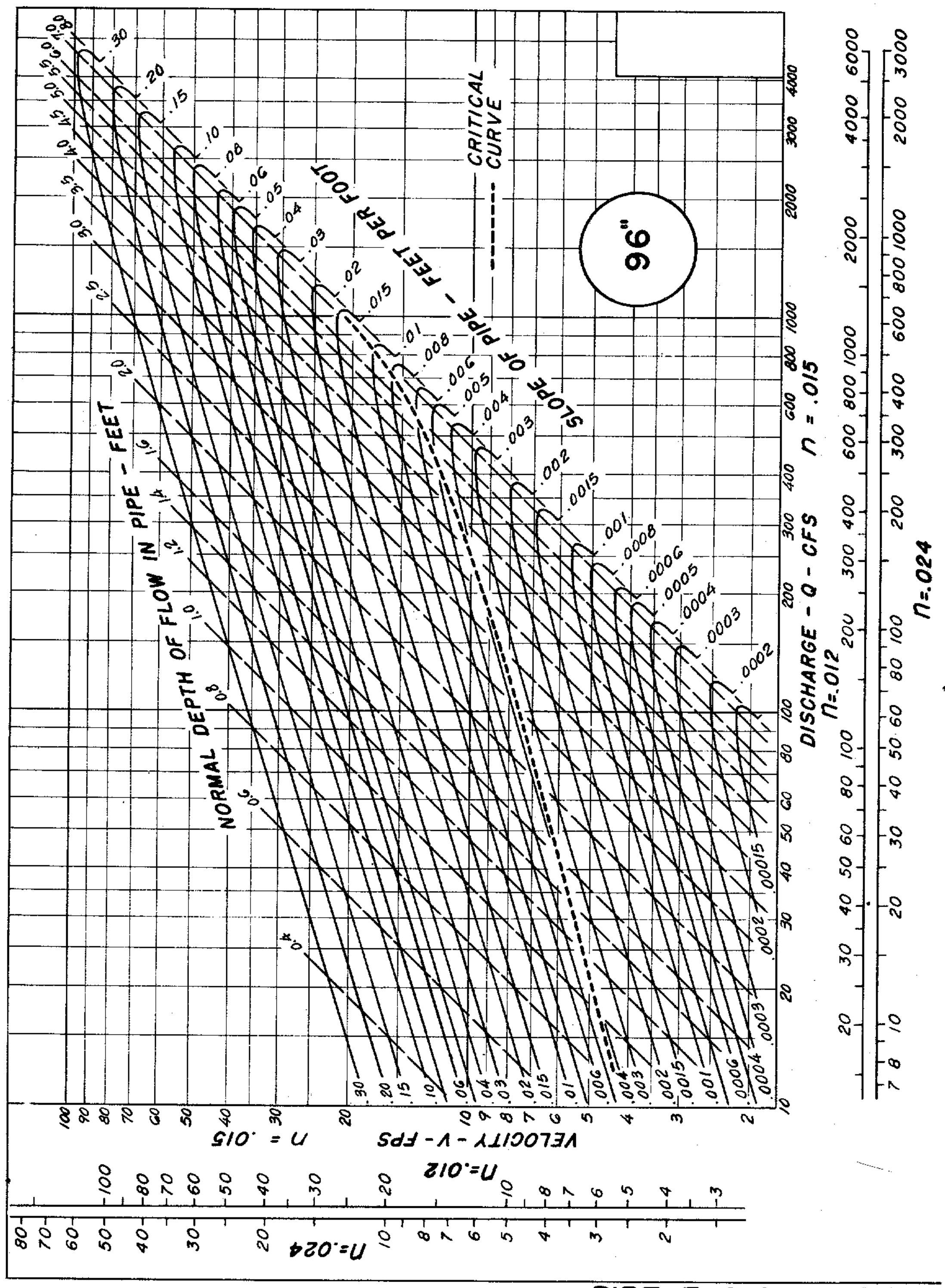
PIPE FLOW CHART
72-INCH DIAMETER

Chart 7.17



PIPE FLOW CHART
84-INCH DIAMETER

Chart 7.18



PIPE FLOW CHART
96-INCH DIAMETER

Chart 7.19